TRANSACTIONS.

546.

WIND BRACING IN HIGH BUILDINGS.

By Henry H. Quimby, M. Am. Soc. C. E.
Read June 1st, 1892.

WITH DISCUSSION.

The use of iron and steel in the construction of fireproof buildings has developed a new type of structure, which calls for the application of the principles that govern the designs of bridges, etc. Technical and other journals, by devoting much space to descriptions of the steel skeleton type of building, have recognized the importance of the subject and some have published good rules for the guidance of designers. Many buildings are now in course of erection in different cities which are examples of this method of construction, and these afford an opportunity to compare the designs and to see how widely the practice of architects varies. One who does compare them is forced to the conclusion that the ideas of the designers differ not merely in regard to details, but, apparently, either in apprehension of the amount and operation of the forces to be resisted, or in their faith in different materials.

The principles that should be observed in designing metal structures are so generally well understood, and there is such a substantial agreement among engineers as to their application, that we may reasonably expect to find every important iron or steel frame building de-

signed in accordance with them. The greatest bone of contention in
this skeleton has long been the use of cast iron, which has been in
large measure abandoned as material for bridges, but which is still
extensively, though decreasingly, used for columns in fireproof build-
ings. There is occasion for an equally vigorous discussion of the
relative merits of hollow tile walls and iron or steel rods as vertical
bracing in lofty structures.

The sole dependence of some architects for lateral stability in their
buildings is on the ordinary partitions, weakened for such a purpose
as most of them are, by doorways through them, while others intro-
duce stout iron rods or braces.

There will be little question of the sufficiency of brick partitions if
there be many of them, when the width of a building is a large pro-
portion of, or is equal to, the exposed height, and the foundations are
firm and unyielding or not subject to disturbance; but a building of
great height even with a good breadth, if on yielding bottom, should
be efficiently braced with elastic metal.

In the type of building mentioned, the columns carry, not only the
floors and partitions, but the exterior walls, which a writer on the sub-
ject recently called "mere curtains to shield the interior." This idea
of their utility is measurably correct if the metal structure is provided
with efficient vertical bracing, but if, as in some cases, the bracing is
omitted or inadequate, masonry of some sort must be depended upon
for lateral stability. As each story of the walls is supported on the
girders of its own floor and carries no load from above, the walls are
independent of the height of the building and may be only 13 inches
thick throughout.

An office building recently erected has seventeen stories above the
pavement, giving it a height of 200 feet, and is only 60 feet wide.
The party walls, which are abundantly able to stiffen the building in
the direction of its length (180 feet), are 18 inches thick, presumably
because of municipal requirements intended to prevent the spread of
fire and contemplating the support of joists. The rear wall and the
walls of a 25 x 80-foot recess, or court, at one side are 13 inches thick,
but are little more than window frames because of the needed provision
for light. The front from the third to the fifteenth floors is also brick
and has two bay windows, the walls of which are 13 inches thick, but
being bowed and of trifling width between the windows, they would

offer very little resistance to a lateral force. The lower two and upper three stories of the front are of stone and terra cotta respectively. The vertical bracing consists solely of the interior partitions, which are 8 inches thick, built of hollow boxing tile with four webs each $\frac{9}{16}$ths inches thick, giving a total net thickness of $2\frac{1}{4}$ inches. At four points 40 feet apart these walls, in a space 17 feet wide, are continuous and without doors or other openings from the third floor to the roof, being apparently the main reliance for lateral stability, most, if not all, the other partitions being greatly weakened by passages and windows.

The building towers above all its neighbors (the tallest in the vicinity being about six stories high and one immediately adjoining only five stories), a fact of moment in estimating the force of the wind. In the same city, a few blocks away, is a building partially completed, designed to have the same number of stories as the one first mentioned, but with a width of about 150 feet. It has provision for vertical bracing in the shape of double 6-inch eye-bars and 15-inch heavy channel struts, contrasting very sharply with the other.

In both buildings the columns of each tier abut against those of the lower tier, with an intervening plate which forms a seat for the floor girders. The tiers are bracketed together with angle iron or bent plate lugs, a detail that is sufficient to prevent lateral displacement, but, because of the elasticity of the bracket in bending, and the large ratio of height to base of column, contributes very little to the rigidity of the structure. This will be seen by computing the stiffness of the

Fig. 1
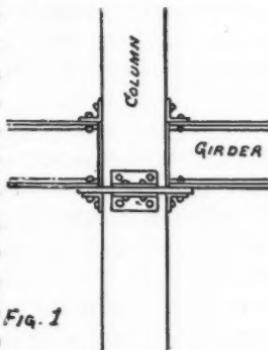
brackets, and by considering that the workmanship is never so perfect that a number of columns could be piled up end to end, without fastening, like children's blocks, which fact indicates that as the columns are always necessarily plumbed—temporary adjustable braces being used if there are no permanent ones—the faces are not all in perfect bearing, more especially as the plates between them are not planed and are consequently somewhat uneven, a condition which presumes some initial bending in the brackets and consequent slight initial lateral strain on the bracing.

The action of the wind against the side of a building produces the effects of overturning and shear, both greatest at the highest point of external resistance, which is the roof of an adjoining building, if there be any, or otherwise the surface of the ground. The overturning or the lift on the windward side is likely always to be less than the resistance of dead weight, but the shear is liable to be overlooked and is probably the immediate cause of the collapse of most of the buildings destroyed by wind. In the type of structure under consideration, the shearing action tends to topple the columns and crush the partitions or rupture the bracing, all in one story, as indicated by dotted lines in Fig. 2, which represents a front elevation of the narrow office building referred to. The column fastenings described are not stiff enough to prevent a slight movement of the tops of the columns, which can be firmly held by the bracing alone. If this bracing is mortared work its cohesiveness is liable to be gradually destroyed by severe vibrations or many successive impacts of pressure ; and if once its hold is loosened, its deterioration will be rapid.

The wisdom of depending on these tile partitions in exposed buildings is doubtful. The crushing strength of the material as tested in small pieces or single bricks, while varying according to the composition of the clay or process of manufacture, in no reported

Fig. 2

case much exceeds 7 000 pounds per square inch of net section. Inquiry of manufacturers has failed to elicit any information as to the strength of a complete wall, and as these tiles are easily broken in handling and laying, they are far more liable to serious and destructive flaws than iron or steel, and, consequently, any "factor of safety" adopted for their use may, with more reason than in the case of structural iron, be called a "factor of ignorance."

What intensity of wind pressure it is proper to assume in designing
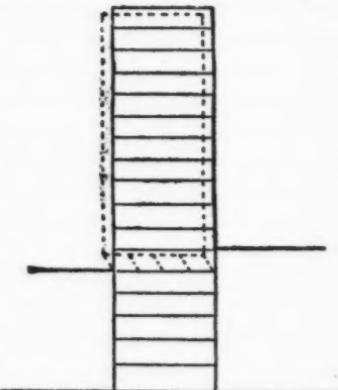
a large and high structure, is an important question.   The forces
which we have to deal with here are not, as in other departments,
limited and known, and the practice must be, to some extent, empirical.
The experiments of the Forth Bridge engineers, and also those made
by builders of wind engines, show conclusively that the pressure per
unit of surface is less over a large area than over a small one, and prob-
ably the ascertained proportion of decrease will fairly apply to the
large surface exposed by a great building.   But the many instances of
extensive and not lofty brick and stone buildings blown down are
warnings against a too liberal allowance here, for a downward current
of air will be deflected by surrounding low roofs or ground and accu-
mulate intensity.   The writer remembers a case of the demolition of a
window by wind near the ground, which occurred at a moment that he
happened to be looking at the particular spot.   The locality was such
that the force of the wind might become, and doubtless was, concen-
trated.

We do know that the wind sometimes develops an energy which
must be far beyond what has ever been measured, and the efforts of
investigators by means of artificial and confined currents of air, have
failed to obtain velocities and attendant pressure sufficient to account
for some of the feats of the natural article.   While we cannot know,
nor be expected to build against its utmost power, the experience we
have ought to induce us to estimate high velocities of wind and use low
intensities of strain in materials.   From 30 to 50 pounds per square
foot will blow in ordinary windows.   Violent storms not sufficiently
destructive to be dignified with the title of tornado have registered
pressures as high as 50 pounds.   They are rare but possible every-
where, and would probably give, over a large surface, at least 40 pounds
per square foot.

In view of the constant liability of any locality to the occurrence of
wind storms of destructive violence and unknown force, every tall
building should be assumed to be subject to a wind pressure of 40
pounds per square foot of exterior wall surface, and be braced to resist
this with iron or steel rods or stiff braces strained not over one-third
their ultimate strength.

The stability of the individual columns in a framed structure is an
element of resistance of considerable value if the connections are rigid.
Even with the connections described above, much ultimate resistance

can be counted on from the bracketed fastenings and dead load, the ratio of base to height of each column being commonly about 12; but because of imperfect workmanship referred to above, they may at first act with, instead of against, the destroying force, and their resistance be developed only after that of the partitions is overcome or impaired. Wherever adequate rod bracing is not employed, the columns should be joined together by complete splices, making each column a unit throughout the whole height of the building, and then failure could only follow the bending or breaking of the body of it at two points.

The foregoing observations have been confined to the effect of wind, but any force operating to produce violent or frequent shocks and consequent vibrations of buildings or foundations, should be carefully regarded and as far as practicable provided against.

Close proximity to the track of a steam road, elevated or surface, but more particularly the elevated because of the concentration of span loads, may subject a building to destructive vibration, and the pulsations of engines or machinery, dynamos or elevators, are even more likely to be a serious menace because of closer contact and more violent shocks. The introduction of isolated electric lighting plants and elevator machinery makes the subject worthy of thorough study, and the possibility of future construction of elevated railway lines through cities, is a legitimate consideration in designing towering structures, particularly in places where foundations are laid in yielding soil like that of Chicago, where settlement must be looked for and where it is probably hastened by vibration impacts. In such localities dependence on hollow tiles for lateral stiffness may be perilous, because light masonry readily succumbs to repeated shocks, and any unequal settlement is certain to injure such walls to some appreciable extent and probably enough to impair their efficiency in the direction of rigidity. A certain high building on such soil was found before completion to have settled unequally several inches, and consequently was seriously out of plumb. Fortunately it was braced with adjustable rods which were used to draw it back into a vertical position, and because of these rods no alarm is felt for it. The unequal subsidence was attributed to an injudicious proportioning of the foundation areas under the different columns which sustain varying ratios of live to dead load. The necessity for the exercise of judgment in determining such proportions is an intimation of empiricism in the business of

designing, and is an additional argument for avoiding uncertain materials in construction.

Another force that is entitled to respect, but is usually ignored in our latitude, though liable to be active at any moment and most powerful to destroy, is earthquake. A notable building in San Francisco has iron plate web bracing from top to bottom, designed as provision against seismic disturbances, which are more frequent on the Pacific coast than in the East, though rarely severe. No part of the country can claim immunity from them, as the experience of a few years past shows, and even slight shocks are destructive to brick masonry of low buildings, while the effect of the undulations of the earth's surface increases rapidly with the height above the ground. The only absolute security against danger from this source is in a system of bracing with some elastic material of positive strength that will so unify a structure that it will hold together even to the point of being overturned bodily, a quality which an enthusiastic writer claims off-hand for steel skeleton buildings in general, but which is probably not possessed by any yet built. A reasonable degree of security against injury from any of the causes mentioned can be obtained with the above suggested provision for wind strains, care being always taken that all details, fastening of bracing, etc., are equal in strength to the main members and competent to properly transmit components of stress.

The buildings which are probably most deficient in wind bracing are those used for manufacturing. They are not as high as some others, although in cities they are increasing in height with the value of the ground, but they are often isolated, and always have large floor areas without partitions, and are stiffened only with small gusset braces or knees which are sometimes inadequately fastened. The regular vibrations of the machinery loosen the hold of the floors upon the walls, and set up destructive movements that make it not strange that when an occasional tornado finds one of these structures in its path, a disaster with appalling loss of life results.

This paper is submitted for the purpose of calling attention to some common faults in designing important works, with the hope that interested members of the profession will record, in the shape of discussion of the points involved, their judgment and observation, to the end that more uniform and improved methods of construction may prevail, a result that will certainly follow a thorough ventilation of the subject.

DISCUSSION.

J. P. Snow, M. Am. Soc. C. E.—If architects and architectural engineers would use built sections of plates and angle irons for their large girders instead of the conventional rolled beams, they could make much more efficient connections with their columns than is usual in ordinary building construction. The use of cast-iron columns with metal floor beams cannot be called good engineering, unless the columns must be exposed as in halls. If we have friendly partitions in which to hide the posts, wrought iron is cheaper and gives us much better facilities for making good connections with the floor girders. With built girders and wrought-iron posts such joints may be readily designed as will give a fair amount of rigidity in those cases where regular transverse bracing would seriously interfere with the necessary openings in the partitions.

A feature that I once used in a design for a high building may be worth describing here. The building was 48 feet wide, and eight floors, each figured for 150 pounds per square foot, together with the roof, were to be carried by the frame so as to leave the floors below clear for halls. I used two main columns of Z bars footing on the walls and inclined inward so as to be 10½ feet apart at the roof. The main floor girders were channel shape, in pairs and riveted across the sides of the posts. The floor stringers were rolled beams headed into the girders. The walls in this case were sufficiently heavy to carry the floor beam weight delivered to them by the ends of the beams. The subjoined sketch shows the style of framing adopted. The inclined posts act to resist an overturning tendency like the legs of an ordinary trestle. The floor girders being riveted or bolted across the posts, make a very rigid and convenient joint. The diagonal members are inside the girders and are quite light; they were arranged in strut form to clear a wide circular-topped central corridor. If doors could be otherwise arranged, a tie system would probably work up lighter. A possible objection to this style of post is the thickness of partitions required to cover it. In defence I submit that a structure must have a skeleton somewhere to sustain it, either on the outside like a crustacean, or inside like a vertebrate, and if the outside walls are reduced to "mere curtains," as quoted by the author, we must be allowed space for a proper skeleton within.

The danger from earthquakes, though remote, should be considered in the design of high structures. There is probably no likelihood in our country of such terrific shakes as occur in the tropics; but such as Charleston, S. C., experienced in 1886, may occur anywhere, and would
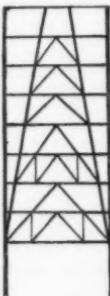
not endanger life, even in the highest buildings, if the frames were properly designed. It was my fortune to have an opportunity to study the effects of the Charleston disaster pretty fully, and the fact was apparent that structures there, which were light at the top and were fairly well tied together, suffered but little. On the other hand, structures that were top-heavy, especially buildings with heavy over-hanging cornices, were sadly demoralized. This seems to be a natural result of the laws of stability. If the inertia of the top is too great for the attachments and fastening to overcome, the top will, of course, be left unsupported when the lower part moves. I do not think transverse bracing is so much needed in case of earthquake as good connections and thorough fastening of all parts together, and a systematic reduction of weight from the bottom to the top; care being taken to avoid all heavy cornices and projecting front ornaments at the top of the building. The mason's art of simply laying one piece on top of another is the very worst style of building that can be adopted for earthquake resistance. The stone veneer of a building with a metal frame would probably be tumbled into the street by a moderate shock unless it was practically disconnected at each floor and made an integral part of the frame by anchors and supporting girders.

Mr. FOSTER MILLIKEN.—Not being a member of this Society, it is with some reluctance that I take part in this discussion, but I hope that my experience in certain directions may be of interest to others. I would speak particularly of the connections of columns at floor levels, because these are their weak points when considering wind strains.

When cast-iron columns with wooden girders were first used for buildings, the round or oval cast pintle was adopted. This was not connected in any way by bolts or rivets to either column, and its equilibrium depended on the wooden girders holding it in line, much the same as a child would pile blocks up and steady the pile with its hands. After numerous accidents during and after erection, the pintle was cast as part of the upper column. This was like taking one of the "blocks" out of the pile at each level, but still accidents occurred. Subsequently, the line of joint was made above the floor, and the pintle cast on the lower column, the connection to the upper column being made by flange bolts. This connection answered for a great many years, and is still largely used, but the days of cast-iron columns for all but buildings of a few stories with short shafts are numbered. Owing to the many faults of cast-iron columns, which it is not necessary here to specify, many engineers and architects refuse to use them for important high structures, and wrought iron and, more recently, steel columns have come into very general use.

The question naturally comes up as to the connections of columns at the floor level. Iron workers followed at first the practice for cast-

Girder.

Girder.

Faced Joint.

Faced Ends.

Faced Joint.

View of Pintle.

I. Beam
when used as strut
in place of 2 channels.

Loose angle on bot. flange to take
arching if necessary.
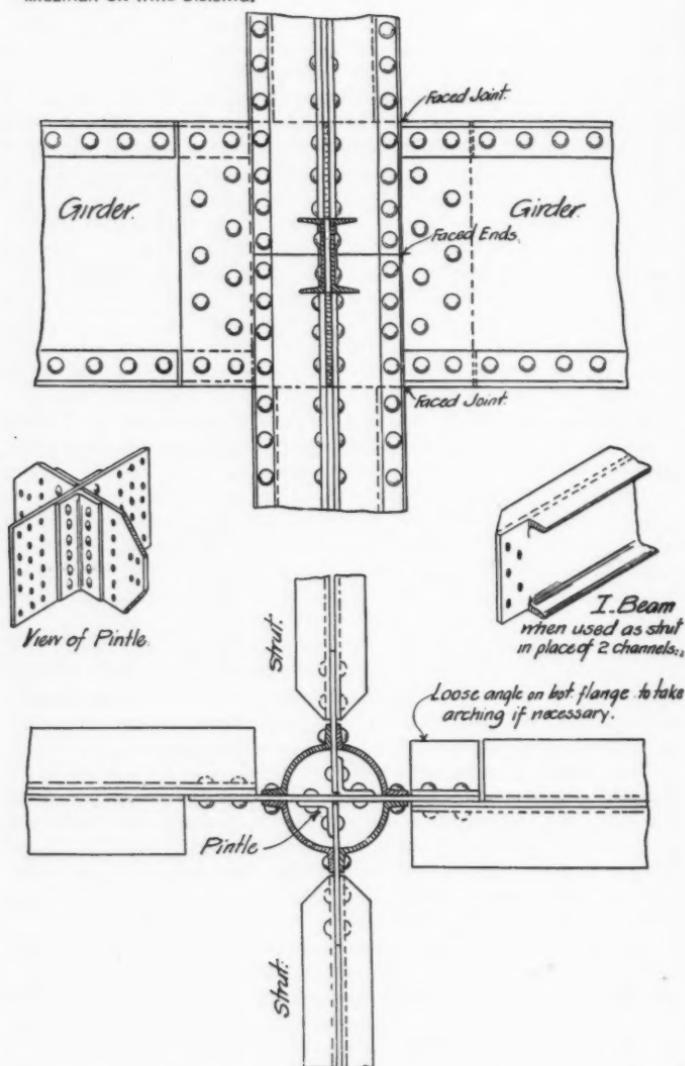
Strut.

Strut.

Pintle.

Cross Pintle Connection – showing method of
connecting Girders and struts.

iron columns, using a bracket near the head of the column for the support of the beams or girders and making the joint in the column above the tops of the beams, or just below the floor level. In some cases the joint was made at the underside of the beams ; but this, except in certain cases, necessitated putting up two tiers of columns before the beams of the floor below could be put in place. Either of these forms is open to radical improvement, as can be clearly shown by reference to an entirely new and different form of connection. The connection illustrated in Plate XLIV is adapted for use with the Phœnix column and is called a cross pintle. Similar forms may be designed for other styles of columns.

I would classify under the following headings the main points constituting the perfect joint :

1st. Continuity of columns from cellar to roof.

2d. Proper connections for load and distribution of loading on the column.

3d. Facility of connection for wind bracing.

4th. Ready alignment of column.

5th. Simplicity of design, thus facilitating erection and minimizing error.

6th. Cost.

1st. Continuity of columns. An ideal column would be one that had a uniform taper and whose section varied from floor to floor as the loads increased. In the design presented we have a column that meets these requirements, for as soon as the connection is made and riveted, the pintle forms a splice or fish joint of the very best kind in which the joint is stronger laterally and vertically than any other part of the shaft. Opportunity for change in sectional area is provided at each tier. There is no leverage to tear the joint asunder, such as there is in any flange joint. All that is necessary to separate a flange connection is to strip the bolts or pull the rivet heads off, especially where the joint is above the floor, in which case the floor beams offer no brace to the foot of the column. The effective length of a column with the cross pintle connection is the distance from the floor level to the ceiling only, thus reducing the effective length.   The ends are in every sense of the word fixed.

As to the second point, the bearing of beams and girders on columns. In the type of connection here presented, the cross pintle is simply a continuation of the webs of the girders clear through the column, distributing the load to all parts of the column, and overcoming to a large extent the tendency of eccentric loads to bear only on a part of the column and thus tend to bend it. The connection of a girder to a column is much more satisfactory when made through the web than through a bracket under the girder flange. Brackets are undesirable, as they interfere with the floor and ceiling finish of a room. The cross pintle itself

transmits its load directly, that is, by actual contact with the filler bars below it, in addition to which the rivets in the column flanges act in double shear.

As to the third point, the arrangement for wind bracing. While this connection would not do away with wind bracing altogether, in a narrow and very high building, yet it can be seen at a glance that where columns of the old style would need wind bracing, this form would in many cases answer without, owing to the very rigid connection of the column to the girder and because the column is continuous from cellar to roof. Where wind bracing is found necessary I would recommend that the cross pintle be made deeper than the girder, and the diagonal rods be connected by pins directly to the pintle plate. This connection would also offer greater resistance in the case of uneven settlement of columns.

As to the fourth point, the alignment of columns. Every one who is familiar with construction knows how much time it requires to "true" columns and align them, and what a great saving it is to have a column that aligns itself. The upper column fitting over a pintle must necessarily align itself whether the workman is careless or not. This brings us to the fifth point, simplicity of design so that ignorant workmen cannot go astray. The writer has often seen columns designed with the utmost care, with ends all carefully faced, set up on nails, bits of slate, sheet iron or anything convenient, so as to correct the error made in the column already set up. It is quite evident that such a column will not safely bear the load it was designed to carry. The connection here shown cannot be tampered with, as the columns will not then fit together. I have always had more or less trouble with columns with brackets; the brackets are often bent and in some cases entirely broken off either in transit, or in handling them while in process of erection. The columns fitted for cross pintles are, when shipped, simply a shaft free from all brackets or projecting plates.

As to the sixth point, the cost of columns is certainly reduced by this system. The greatest expense in shop work is work that requires forging. This plan of connection is absolutely free from bent plates of any kind. Further, the plates are always of such size that they are easily procurable. The work is of the very simplest kind, while the weight of the connections themselves is from one-half to two-thirds that of the ordinary cap and base plate connection.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—Mr. Milliken has modestly omitted to mention that this pintle connection, which he has described, is being used in the columns of the large power-station of the Broadway Cable Road at the corner of Broadway and Houston Street, by Post & McCord, contractors for the iron-work, where, owing to the difference of elevation between different parts of the first floor, some of the pintle plates are more than 8 feet in depth. In this floor

the pintle plates only extend a very short distance above the joint, so, that, in case the second floor columns should not be set for a while, the pintle plates would not be exposed to injury or be in the way.

Returning a little nearer to the subject of the paper under discussion, and considering the vibration of high buildings due to wind or other causes which may be prevented by suitable bracing, I would say that the worst instance of the kind that has come under my notice was in the six-story brick building, the top floor of which was formerly occupied by the *Engineering Record*, where we were really shaken out of our quarters and forced to move out. The building was very much cut up by windows in front, so that it had no lateral stiffness and the sway on the upper floors was such that if you sat with one leg crossed over the other, you could not keep it still. I know one gentleman who was made actually seasick by the incessant vibration, which shows the extent to which the building was shaken. It was due, undoubtedly, to the printing presses, where the type-form and bed traveled rapidly backwards and forwards, and had their motion suddenly arrested at each end of the stroke.

This swaying of buildings is felt sometimes at great distances from the force that starts it. In the New York Steam Company's building on Greenwich Street a large blower shook the building, although a very heavy one, and the blower was placed right in a corner where you would expect the building to be stiff. The tremor was felt to an annoying degree through and to connecting buildings on the same block as much as a hundred feet away.

I might mention that in erecting this same building the heavy cast-iron bases for the columns were very carefully leveled; but, when we had the first tier of columns set, we found that their tops were not level at all. We could not understand how the two faces of the turned columns could fail to be parallel. We investigated the matter, however, and went over to the shop where the columns were made, and found that they had put them on their lathe shears, and blocked them up, and, clamping the columns fast, had started a cutter attached to a face plate at each end. As the shears were of wood, and exposed to the weather, they naturally got out of line in course of time, so that the face plates were no longer parallel, and, of course, the finished ends of the columns could not be either. This incident shows that unless you know how your columns are turned, you cannot be sure that their finished faces are parallel merely because they appear to have been done in a lathe. I tremble to think how many columns went through that old machine before we had it trued up, and are now carrying heavy loads on one edge only of their bearing surfaces.

To return again to our wind bracing, the only building in which I

have observed anything of the kind is now going up on Vandewater Street, near the bridge, which some of you may have noticed. Its walls are iron frames filled with windows, while the narrow rear wall is paneled with brick. This wall alone has diagonal bracing in it, perhaps because it is so narrow that it may be supposed to need it, but more likely because the windows would have interfered with it elsewhere.

In a building, also in plain sight from the bridge, put up recently at the corner of Pearl and William Streets, the iron-work of the interior was, as is not uncommon, carried up two or three stories ahead of the brick walls, and a derrick used for hoisting material had its back guys fastened to the upper floor, while its foot rested on the floor below with nothing to resist the pull of the guys, except the stiffness of the column connections. It did not pull the top floor over, but I wondered what kind Providence prevented it. In such a case good wind-bracing would have rendered important service during erection.

J. FOSTER CROWELL, M. Am. Soc. C. E.—There is an eleven-story office building now in course of erection at the corner of Broad and Beaver Streets, in this city, which, if the criticisms in this paper are well founded, exhibits great boldness on the part of its designer. It stands on a pile foundation, and is carried on cast-iron columns from base to roof throughout. The exterior walls are of masonry, serving as curtains merely, and do not extend below the street level, so that the first tier of columns is without vertical plane-bracing.

The arrangement of columns and floor connections are different from the types just described. The floor girders are secured to the tops of the columns, which pass between them to the upper flange, resting on brackets or lugs cast on to the columns. Each tier of columns rests directly on the tops of the tier below, and is held in place, laterally, by flange bolts, which evidently have no adequate relation to diagonal bracing.

The several stories are simply a succession of "tables" piled one on top of the other. There can be little or no rigidity in the connection between the feet of the columns and the girders, and the architect probably relies on the table construction, in addition to the masonry filling in the exterior walls and partitions, for bracing against overturning. In case of unequal settlement of column foundation, it would seem as if there might be serious trouble in this type of building, due more particularly to the use of cast iron for columns.

In regard to the suggestion in the paper, as to whether special provision should be made for earthquakes, I venture the opinion, speaking, of course, of buildings in this part of the world, that if a building is secure against the combined action of gravity and of the high-wind forces of this vicinity, there need be no apprehension that it will not

withstand the earthquakes that are likely to occur in these latitudes.

I once had the experience of being thrown out of bed by an earthquake in Nicaragua, and although I cannot boast of very much knowledge on the subject of earthquakes, yet I think it is a good deal of a bugbear, a view which I think is borne out by the experience of the San Francisco builders, whose more modern constructions resemble those of other localities, with rarely any special consideration for earthquakes in their design. Even in tropical countries, the general earthquake does not appear to be of a character to affect modern massive buildings. I am aware that there have been massive buildings destroyed by earthquakes, but if they had been constructed as our best modern buildings are, I doubt if they would have been.

The earthquake effect that is propagated to regions remote from its origin, seems somewhat in the nature of a sharp and sudden stroke on the lower side of the earth's crust. It seems to be local, largely ; it does not as a rule extend over a large area of country; but wherever it occurs it is the manifestation of impact from below, and does not appear to move laterally along the surface, but consists of a succession of shocks. The seismic records, I think, indicate that this condition is the usual one for propagated earthquakes.

My own little experience confirms the supposition. It occurred when lying in a peculiarly constructed camp bed, made by driving four stout corner stakes into the ground, lashing side-bars and cross-braces to them, and then stretching very tightly a canvas hammock, with a cordlacing all around, so that the bed was very elastic, like a drumhead. When the earthquake, which was some forty-five seconds in duration, manifested itself, several of us who were occupying separately such beds, were thrown out; the shock came distinctly from below. We were "snapped out," so to speak, and landed on the ground, right near us. We were a few yards from the San Juan River ; a large tree which had overhung the bank was thrown down.

To return to the buildings, I do not, as I have said, consider the earthquake contingency to be a very serious matter in this country. Of course, we can imagine earthquakes against which all builders would be impotent, but buildings properly designed to withstand tornadoes and their loading will, I think, stand any ordinary earthquake.

HENRY B. SEAMAN, M. Am. Soc. C. E.—The lateral stability of buildings has received much less attention than have the provisions for vertical loads. Roofs are designed for strains which, in their combination, rarely, if ever, occur, while the sides are proportioned for the most casual wind pressure, or for none at all. The pressure of 40 pounds per square foot, for which most roofs are designed, would probably collapse the majority of structures on which they are placed. The recorded pressure at Central Park protected, as it is, by New York City,

is 38 pounds per square foot, and pressures far exceeding this have been recorded in more exposed localities.

While these pressures may be confined to small areas, it should be remembered that buildings are not capable of the accurate adjustment that may exist in bridge structures, and we cannot depend upon the same simultaneous resistance of the different parts. The practice of specifying light pressures, and depending on a factor of safety for the requisite strength—as is the custom with many constructing firms—is objectionable. The factor of safety has many structural defects to provide for, and in this class of work is not prepared for additional loads. There is, however, a constant tendency in this direction by those whose interest it is to save first cost, at the sacrifice, if necessary, of permanent security.

GEORGE A. JUST, M. Am. Soc. C. E.—It seems hardly necessary to calculate buildings to resist a wind pressure of 40 pounds per square foot of exposed area. Danger from wind exists mainly during construction, while walls are still "green" and the building remains to be "topped out." The majority of accidents probably happen before this stage is reached.

Buildings have been erected in this city—and, no doubt, elsewhere—with such a disregard to resistance to wind, that the conclusion seems inevitable, that they remained intact only by the operation of forces that are ordinarily ignored by engineers in their calculations. However, the architect and builder rely upon these forces and in the end they seem sufficient. We admit this is not good practice or good theory.

If Mr. Quimby, in his paper, had submitted to us a system or method of bracing meeting the requirements of architects, satisfying at the same time all technical considerations, he would have led us a step in the right direction. But a provision for 40 pounds per square foot applied uniformly over the entire exposed area of a wall or building seems more than necessary. On the other hand, the destruction of buildings is often aided by the disintegration of green walls, caused by the vibration of hod-hoisting engines and other appliances extensively used in the erection of modern structures. In buildings of the class under discussion, the careless methods of the "architectural iron man" should certainly be supplanted. The same reason for making the columns continuous by means of ample splices exist here as in the case of "bridge-bents" or "viaduct-towers."

The detail shown by Mr. Quimby in Fig. 1 of his paper, and one that is generally used since the introduction of wrought columns in buildings, is a poor one. It prevents good splicing and girder connections, and makes the erection of the next higher tier more difficult for the "setter." On this point the "rolling-mill engineer" could well follow the less professional "iron-man" who always attaches his

girders and beams to the head of the lower column and makes his joint at or above the floor line. I happened to be connected with the erection of the first and some of the subsequent buildings of this class in New York. The first, known as the "Tower Building," I declared unstable as it was being "topped-out." Mr. Theodore Cooper, M. Am. Soc. C. E., subsequently sustained this position in a report to the architect.

This building of eleven stories, with basement and cellar, reaching about 116 feet above the curb, with a wing or frontage of only 21 feet 6 inches on Broadway, was constructed with the usual cast columns spaced 13 to 19 feet on centers, with longitudinal and cross beams in the floors. The connections of the latter to the columns were made in the careless manner usual to buildings, so that for purposes of calculation it was deemed wise to consider the side or gable walls as virtually independent of each other. As a result five trusses (Fig. 1) were introduced into the narrow part, or Broadway wing, of the building, transversely, and extending from the cellar to the roof. The columns of the south wall were utilized as a chord. The other chord consisted of double L irons placed 13 feet away at the line of the hallways, which of course could not be closed up. This new chord was attached to the intersecting floor beams at each story, the web members by panel plates to the angle-chord, and by tap bolts and bent plates to the cast columns. While the value of the latter connections depended mainly on the skill and care exercised by the workmen, it seemed at the time the only means
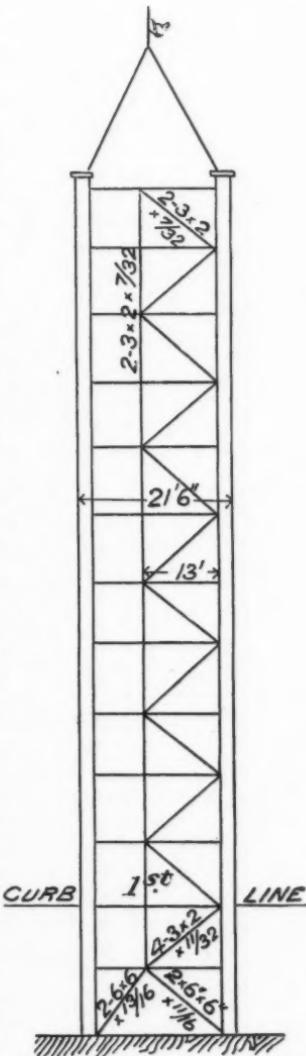

FIG. 1

to an end.   In the worst case the loads assumed induced strains that required chord sections at the bottom of two 6 x 6 x $\frac{13}{16}$-inch angles, and end webs each of four 3 x 2 x $\frac{11}{32}$-inch angles.   These small-legged angles were used so as to confine the iron within the partitions, none of this trussing being visible in the completed building. The trusses were spaced 13 to 25 feet center to center, to meet the position of the wall columns already in place.

Mr. SEAMAN.—Mr. Just remarks that he does not agree with the provision for wind pressure of 40 pounds, and then speaks of buildings that were too light.

Mr. JUST.—I believe we can use, at least in this vicinity, with safety, a lighter load than proposed, because the U. S. Signal Service here has only once recorded a velocity of 84 miles an hour, or about 35 pounds per square foot.   They consider 60 miles an hour, or 18 pounds per square foot high, and record more often only 50 miles or 12$\frac{1}{2}$ pounds per square foot; because factors omitted in our calculations do nevertheless aid the stability of a structure, and because, as Mr. Quimby points out, experiments with anemometers seem to prove that high pressures are local and not extended over large areas.

Mr. SEAMAN.—Then you want to provide for 35 pounds ?

Mr. JUST.—No, I believe 30 pounds will answer the requirements of safety.   It will be recalled that in the experiments bearing on this question, made during the construction of the Forth Bridge, a revolving gauge, a small fixed gauge, and a large fixed board gauge (15 x 20 feet) were used.   The large board gauge was further provided with a small gauge in its center, and with another at one of its upper corners. The maximum results obtained in all that time were—

Date.	Revolving Gauge.	Small Fixed.	Large Fixed.	Center Large Board.	Upper Corner Large Board.
Gale March 31st, 1886..............	26	31	19	28$\frac{1}{2}$	22
Gale January 25th, 1890...........	27	24	18	23$\frac{1}{2}$	22
Highest Record....................	35	41	27	....	....

i. e., over this limited area of only 300 square feet no higher pressure than 27 pounds was ever recorded, while this bridge was designed for twice 56 pounds over the whole area of the girder surface exposed, in accord with the rules of the "Board of Trade," yet the report made to this board by a body of engineers in 1881, and upon whose recommendation these rules were presumably based, closes with the remark: "If the lateral extent of extremely high gusts should prove to be very small, it would become a question whether some relaxation might not be permitted in the requirements of this report."

The engineers of the bridge, after their experiments, concluded that

"the higher wind pressures come more in gusts and sudden squalls, than in a steady even pressure extending over a large area." Reference may here also be made to Mr. Shaler Smith's paper on "Wind Pressure upon Bridges," printed in Vol. X of the *Transactions*, in which, while recording local pressures as high as 93 pounds per square foot, he states: "I very much doubt if a direct wind or gale ever exceeds 30 pounds per foot," etc.

A. F. SEARS, M. Am. Soc. C. E.—When I came into the hall this evening, I had no intention of saying a word on the subject of this paper; and if the lessons to be drawn from the discussion were to be limited to the City of New York I still would have nothing to say about it; but as the *Transactions* of this Society are read everywhere and the principles of construction we are supposed to develop here are made use of in every part of this continent, it may be well to correct or rather develop one statement, and say, in connection with Mr. Crowell's remarks, that structures erected for earthquake countries, if built for only one kind of earthquake, would be insecure affairs.

Mr. Crowell has told us that earthquakes are like blows upward from under the crust of the earth. Now, that is only one of three distinct classes of earthquake; it is, however, the most dangerous of all, getting the name in Spanish countries, where such phenomena do most abound, on account of the terror it inspires, of the *Choque de Trepidacion*. This is the shock that knocks down walls.

So far is it from being true that massive structures are not injured by earthquakes, my personal experience and observation is precisely the opposite. I would say it is the massive construction that is injured, that is exposed more than any other to danger. Only a few years ago, you will recall, in 1867 or 1868, the City of Arequipa was destroyed. It suffered more than the cities of Lima or Callao have suffered in similar upheavals. Arequipa is solidly built of stone, while the others are of the lightest possible material above the lower story. Callao has suffered by being submerged, but not by the shaking down of the buildings.

The Peruvian method of building has been evolved from an earthquake experience. Thus, the lower story is a wall of adobe of thickness varying from 2 to 4 feet. This is reckoned safe for a height of perhaps 15 feet, when the circumstances of a church or some other public building make it unavoidable, but it is not desirable to go above 12 feet with a solid wall; after that, the superstructure is of bamboo, whether in churches or other great buildings. What appear heavy towers on the Cathedral and all the imposing structures of Lima, are, in reality, only bird-cage construction, stuccoed with mud and finished with gypsum. This light work is of bamboo from Guayaquil, which being stripped into long narrow lathes, is fastened with wire or raw hide to upright posts of similar bamboo left entire.

In the City of Mexico the walls are of stone, and yet when I was there about seven years ago, nearly all the high church walls, and they are decidedly massive, were cracked through by an earthquake in a very considerable part of their height. So badly cracked were they that many streets were guarded with chains and ropes, to prevent the passage of teams, the jar of which might cause the injured walls to fall into the streets.

Earthquakes manifest themselves in one of three ways. The first, as already described—a blow from underneath the earth's crust; it alone is called a "shock." The second is vibratory or trembling and is characterized accordingly among our shaken-up neighbors as a *temblor*, or trembling. The remaining kind is called a *terremoto* or earth movement and is undulatory.

L. L. Buck, M. Am. Soc. C. E.—Has Major Sears ever discerned any cracks in the old stone bridge at Lima? It was built in 1610.

Mr. Sears.—No; that is a very low structure, very solidly built; almost a solid rock in fact. Some of the towers of Lima are pretty high; those of Santo Domingo, for instance; but they are all of bamboo.

Mr. Buck.—Would not some of those heavy waves crack such a bridge as that?

Mr. Sears.—I should think not. I never noticed anything of the kind. If, however, it had been as high as the buildings in the City of Mexico and built as it is, it would have been broken up as they were.

F. Collingwood, M. Am. Soc. C. E.—Some little time ago I was called upon to examine a building on Pearl Street in this city. This building has stood there for probably more than fifty years. The foundation of the front portion is on sand, and of the rear portion on piles, and the back wall was showing signs of failure. The floors were very heavily loaded with coils of fence wire, and at times the piles of coils would be seen to vibrate very considerably. The building was four stories high, and the rear wall of the upper three stories rested on stone columns in the first story. A crack had developed by the separation of the rear from one of the side walls, the rear wall, having moved outward for about three-eighths of an inch at some points. I took levels on the floors to see that there had been no settlement and then made a wedge of wood and inserted it into the crack, marking carefully the depth to which it could be inserted. In about two weeks I found the crack had widened about $\frac{1}{32}$ of an inch, making it evident that the trouble was progressive. The cause might be one of two, either the effect of vibrations caused by the elevated cars passing close to the rear wall, or the vibrations fromdy namos in the Electric Light Station, which was about three doors farther north. Which of these was the cause I could not tell as I could not be there all day to see, but it was evidently the result of vibration. The remedy which I proposed

and which was effective, was to drill holes through the walls and fasten iron bars with heavy anchors outside the walls, attaching the bars to the under side of the beams of each of the floors. In that way the front walls were anchored to the floors; and, as the floors were securely fixed in the side walls, there has been no trouble since.

H. F. Dunham, M. Am. Soc. C. E.—I once had a brief experience upon the top of a cliff that may be of interest in this connection.

The cliff was about 500 feet in vertical height, in this respect resembling a Chicago structure. It was three-quarters of a mile in length. From the base of the vertical portion of the cliff there was a slope of 3 or 4 to 1 for several hundred feet to the valley below. At the time, the wind was blowing very hard from a direction at right angles to the face of the cliff. There was nothing to break the force of the wind within a distance of many miles. Upon the slope at one end of the cliff, and in fact in the valley, it was difficult for a person to keep an erect position without considerable wind bracing, but upon the top and immediate front of the precipice there was no wind to be observed. There was no movement of the air from any particular direction. It was at first puzzling, but upon throwing an object directly upward to a considerable height, it was caught by a strong current, and carried upward and toward the back side of the mountain. A stick thrown in a horizontal direction from the top of the cliff was also carried upward, falling a considerable distance behind me. There was a very strong current in this relation to the cliff.

Now, what effect this upward current had in relieving the face of the cliff from the force of the wind is not known to me, and perhaps could not be very definitely ascertained, but the upward current did completely protect me from any horizontal currents while I remained upon the top of the cliff.

R. L. Harris, M. Am. Soc. C. E.—An example of wind effects, such as the gentleman has mentioned, was experienced by me at the Palisades Hotel in 1876. It will be remembered that this hotel was situated about 200 feet back from the face of the nearly vertical basaltic cliff, rising over 300 feet from the westerly shore of the Hudson River. During a very severe easterly gale, by dint of crawling on hands and knees over the nearly level, open lawn, I managed to force my way to the edge of the cliff. Here, and for 25 feet back, there seemed to be a dead calm; there was no wind whatever in any direction, the wind was above me. The return to the hotel in safety was difficult, but far more speedy than the approach to the cliff.

As to the earthquakes in San Francisco in 1868. My home was then in that city. Some large brick buildings were shaken down, and there was also considerable damage done to stone copings, balustrades, etc. The event caused a great deal of talk, investigation and study among engineers and architects in the way of devising "earthquake-proof"

structures.   What then seemed the best was to give a degree of flexibility to brick structures by running thin iron bands vertically in the walls.

To show how old residents regarded earthquakes, I will cite the case of the wealthy James Lick, the patron of arts and sciences on that coast.   He built the hotel called the "Lick House" on Montgomery Street, thoroughly and substantially by day work, and with especial regard to earthquakes; it was only three or four stories high.   A public reception was given on the day of its opening.   To the surprise of Mr. Lick, the new managing firm opened a finely appointed suite of rooms, telling him it was for his sole use whenever he visited San Francisco· He ejaculated, "For me ! For me !! I would not spend a night in a brick or stone structure in the city for all the hotels; no, you will find me at my ranch, although I have built this hotel with the object that it shall be 'earthquake-proof.'"

JOHN BOGART, M. Am. Soc. C. E.—Most of the discussion this evening has been in regard to what has happened to buildings from vibration.   In this paper there does not seem to be, as I have looked over it, any reference to actual experiences of failure from lack of bracing against wind pressure.   Would it not be well to give in the course of the discussion, references to authentic instances of failure of buildings from lack of bracing against wind pressure, with some record of the character of construction of buildings which have actually failed from that cause.

MENDES COHEN, President Am. Soc. C. E.—It occurs to the Chair that the paper presented is treating of a subject comparatively new. The style of building referred to is one that has only been constructed within the last few years, and probably the experience in the direction to which you refer has not yet been accumulated.   It occurs to the Chair that the author is simply throwing out the idea that such trouble is likely to arise.   If any of our members can offer any light upon any experience already attained, I am sure we would be very glad to hear them.

Mr. CROWELL.—Although the author does not specify cases, he refers to many.   I think it would be well to have him specify.

Mr. BOGART.—That is the point I was trying to get at.

Mr. SEAMAN.—We had a case in Jersey City only a year ago, in which the walls and the roof of the Wells, Fargo & Co.'s building fell down during construction.

Mr. JUST.—The case mentioned by Mr. Seaman, as also that of the Armory at Fourth Avenue and 91st Street, were not cases of "skeleton" construction.   Here, long walls, still green, unbuttressed and unsupported from end to end, failed, because the only permanent bracing intended, viz., the roof purlins and jacks had not been placed in position at the time of failure.

Mr. Buck.—I think that the question of bracing of high buildings is one which most of us know so little about and yet wish to learn about, that we would prefer to ask questions about it rather than to answer them.

There is one question that I have heard frequently, and that is, should not a high building with an iron frame, covered with masonry walls and partitions, be provided with diagonal sway bracing of iron to insure its stability? My answer has always been in the form of an opinion substantially as follows: When a building with a well-constructed iron frame is covered with well-built masonry walls of a thickness sufficient to prevent sudden changes of temperature affecting the iron frame, and the proportion of window space compared with that of solid wall is not too large, I see no advantage that would be gained by the iron sway-bracing; for the reason that the rigidity of the walls would exceed that of the braced frame to such an extent that, were the building to sway sufficiently to bring the iron bracing into effective service, the walls would have become damaged. Consequently, it appears to me that the building should be so constructed as to make the stability against swaying depend entirely upon the masonry or else upon the iron alone.

A strong iron frame of columns to which the iron floor-beams are secured by proper joints, with masonry walls not only covering the outer sides of the iron members, but built in between them to protect them from changes of temperature and keep the columns straight, while the spaces between the floor beams are filled with masonry arches or hollow brick or cement,—would appear to possess all the stability required. The floors, themselves, are sufficient lateral bracing; the columns secured from buckling by the masonry and strut and beam connections will support the floors and their live loading. All the walls and partitions have to do is to bear their own weight, and prevent swaying of the building. They should be capable of doing this effectively.

The question appears to be more applicable to the enormously high buildings which have been projected in some cities in this country, buildings so high that it does not appear possible that the masonry walls, after considering their own crushing weight, can have much efficiency left for the purpose of bracing the frame. For such buildings it would appear best to use the metallic sway-bracing and plenty of it, while the walls should be made of lighter material and merely serve to cover the iron and enclose the building.

Mr. Seaman.—I think the wall edgewise is undoubtedly the stiffer. An example of the flexibility of iron bracing was well illustrated in Philadelphia several years ago. A tall brick building which, with its wings, formed in plan a U, was seriously injured by the vibrations of an engine at the top of one of the wings. The wings were afterwards

most thoroughly braced by a system of rods and struts between them. When the engines were again started however, the cracks continued to increase, as before, and the engines were finally abandoned.

Another instance is that of the iron coal-breakers recently constructed by E. B. Coxe, M. Am. Soc. C. E. These breakers are, necessarily, high, and have a great deal of machinery near the top. The vibrations of the iron building, I understand, are perceptibly greater than formerly took place in the wooden structures, though in neither case has it been a source of inconvenience.

Mr. Just.—A direct comparison such as Mr. Buck suggests can hardly be made in this city. I know at this time no two buildings of similar size and style, the one designed with, the other without, diagonal rods and braces. Large gussets at the intersections have generally been used to secure lateral stiffness. Doors, windows and hallways as a rule interfere with any regular system of bracing.

Perhaps the best-constructed building of this class in this city is that of the Lancashire Fire Insurance Company in Pine Street, designed by L. de C. Berg, M. Am. Soc. C. E. (J. C. Cady & Co., Architects). Built in 1889–90, it rises in ten stories 120 feet above the curb, with a frontage of about 24 feet. The side walls and floors are carried by wrought-iron Z-columns, 12 feet on centers, anchored to the foundation by rods encased in lead pipe. At three different levels in the height of the building, riveted girders are placed in all four faces of the building, well connected to the columns, and by extra large gussets in the front and rear walls. He also introduced at these levels over the floor beams diagonal ties of flat iron. He further recognized the force of the wind by designing his columns and girders in each wall to carry the dead loads, and an additional vertical load of 15 pounds in lieu of wind pressure over the exposed surface.

The architect of the Havemeyer building, at Cortlandt and Dey Streets, Mr. George B. Post, introduced a partial system of sway rods. The building covers an area of about 214 x 50 x 60 feet, and rises about 172 feet above the curb. Assuming that the Dey and Cortlandt Street fronts were braced by these walls and the center of the building by a semi-circular wall or rotunda, built out from the rear wall, he provided, at two intermediate points about 30 feet from each end of the plot, diagonal round rods and turnbuckles between the two columns nearest the front, as also between those two nearest the rear of the building, forming half-bents. These rods, however, needed constant adjustment during construction, since the outside columns built on the same footings as the outside walls sank more than the next columns to which they were tied by the rods and which rested on isolated piers.

While perhaps not germane to the subject, it may be interesting to note the evolution of "skeleton" building in the metropolitan dis-

trict. To secure the greatest amount of light for commercial buildings, architects were first induced to reduce the size of their front piers by building into the heart of the same a line of columns which should sustain directly the loads of the floors, and then by cross-girders connecting the columns, the weight of the front bays. The constantly increasing value of land next led to higher structures, and hence thicker walls. To reduce the encroachment of the latter upon the rentable floor space, columns with "curtain" walls were resorted to. This saving led to the adoption of a skeleton construction in the case of the Tower building in 1888. It may be added that the price reported paid for the plot of the Lancashire building was $106 per square foot. This new construction naturally gave rise immediately to discussion on related questions, such as the effect of the expansion of the metal on the permanency of the building, and the relative rate of corrosion in cast-iron columns *vs.* wrought iron and steel.

The concentrated loads transmitted by the columns to the foundations in this system, together with an inability to extend the footings upon any adjoining property led to some ingenious devices to reduce the consequent eccentric loading and to distribute the weight uniformly over the same.
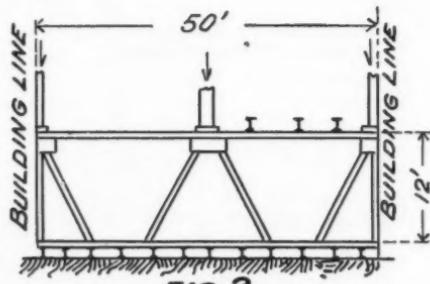
FIG. 2

In the case of the addition to the Western Union building on Dey Street, the architect, Mr. H. J. Hardenbergh, covered his floor with rolled beams, and across the same at certain intervals placed riveted girders of a depth equal to that of the cellar story (Fig. 2). To accomplish the same end in the Hays building in Maiden Lane a cast shoe over the pier with cantilever beams in the floors, carrying the wall columns on the short arm, was resorted to (Fig. 3).

It must be noted that the engineer or architect is often hampered in his design by the "laws relating to buildings"—generally bad, and compiled without the aid of engineers—that may be in force at given times in all large cities.

Thus the Building Law for New York before its amendment in April of this year did not countenance the erection of skeleton structures. A special application to a "Board of Examiners" might, however, secure the privilege. In the beginning such privilege allowed the use of 12-inch curtain walls throughout the entire height. Later on, 12-inch, 16-inch and 20-inch walls were required. The law as now amended demands a 12-inch wall for 50 feet from the top and for each succeeding 50 feet a wall 4 inches thicker, so that a building 200 feet above the curb must have 24-inch walls at this level. Of course, this new requirement destroys in a measure the value of skeleton work for narrow lots. The amended law requires also that girders carrying curtain walls only 12 inches thick shall be placed at each floor level. When 16 inches thick they can be placed at every other story level, and when 20 inches thick the floor beams can rest directly on the walls, but at each story level where no girders exist a line of "ties" must be built "horizontally" into the wall.
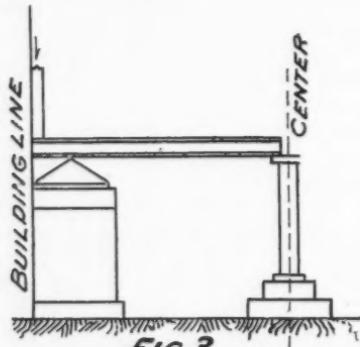
FIG. 3

Under the law buildings can be erected either with walls of "lawful thickness," of "curtain walls" as above, or with "non-bearing" walls which can be 4 inches less in thickness than the "lawful thickness," in which case the walls are carried up alongside the interior iron construction, being simply anchored to the same, and of which style the World building is an example.

After these excerpts from the law, it should be unnecessary to urge the professional engineer to take a more active interest in legislation, pending at any time, on matters which concern him so closely.

In the earlier skeleton buildings the frame was not carried to the roof. The only motive for adopting this style was a saving of floor space, so that when a height was reached at which the law specified walls 12 or 16 inches in thickness, a girder was placed over the top of

the columns. The walls were continued over this girder, the floor beams resting in the walls, just as in a building standing directly from
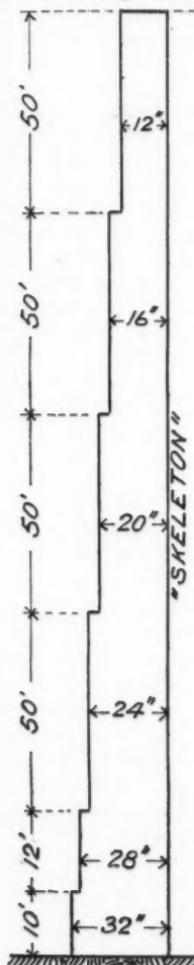
FIG. 4
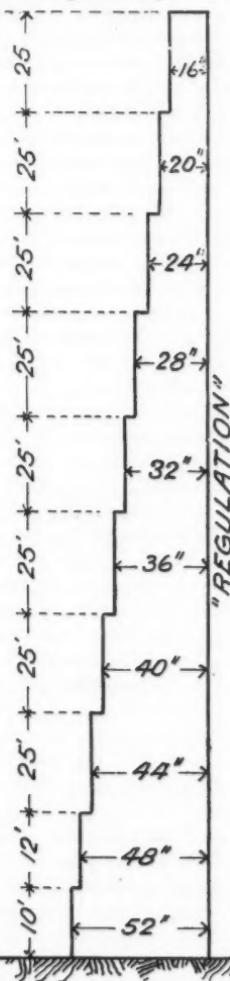
FIG. 5

solid foundations. The Jackson building, with 28 feet 6 inches front on Union Square, is an example. Carried on cast-iron columns placed

15 feet on centers, with cross girders of 20-inch rolled beams, and 10-inch longitudinal floor beams, the frame rises to the top of the sixth story. Here, a girder forms the foundation for a superstructure of brick 20 inches thick for two, and 16 inches thick for three additional stories, a total of 154 feet above the curb. The wall surface here exposed is 184 x 61 feet in height. Figs. 4 and 5 indicate the advantage that still maintains in "commercial" buildings under the new law. They show the required thickness of brick walls for a skeleton building, and for one of "regulation" thickness, 200 feet high above the curb.

Let us take a typical "skeleton" building, 25 feet front, with 12 stories above the curb, each of 12 feet, and a basement and cellar, in which the wall columns stand opposite each other, and are spaced 12 feet on centers in plan. Assuming that the length of the building affords sufficient stiffness in that direction, and that the same is to be secured only against yielding sideways, we could, apart from architectural considerations, place diagonal members through each story, as in Fig. 6, utilizing the cross-floor girders as posts (or ties), and the wall columns as chords.

If each truss so formed be now subjected to a wind pressure of 40 pounds per square foot of exposed surface, we should have panel loads of 5 760 pounds, shear of, say, 38 net tons in the cellar diagonal, a tension of about 146 net tons at the foundation of the windward side, and a compression of nearly 130 net tons on the leeward. The columns and girders in
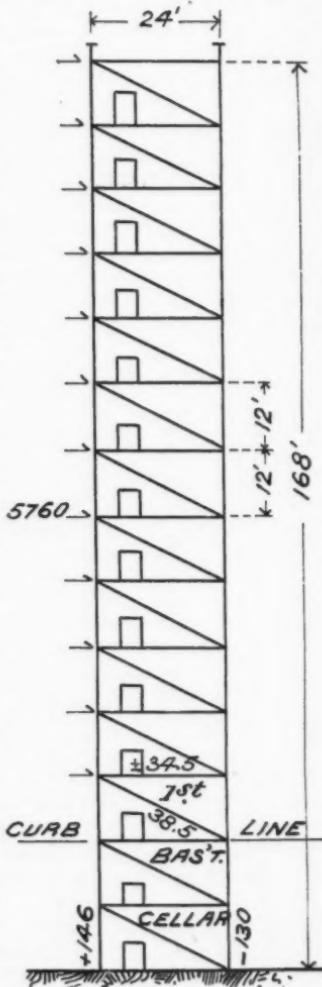
FIG. 6

this side should then be proportioned for this additional compression, and the cross-floor girders of the lower tiers for 35 tons (the upper ones proportionally less), tension or compression, depending on the direction of the wind. All connections would have to be designed accordingly, while the floors would give the necessary "sideways" strength needed in the wall girders. With an assumed load of only 30 pounds, these forces would, of course, be reduced by 25 per cent.; and with a building 120 feet deep, requiring eleven such trusses, the lesser load would effect a considerable economy in the cost of the building.

The dead weight (consisting of iron-work, brick walls, as per the New York law, floors made as usual of hollow flat arches, filling and wood flooring, together with plastered ceilings and walls) on any one foundation is about 440 000 pounds (for iron-work only and walls 12 inches thick throughout, this becomes 322 000 pounds). The tension at the bottom of the windward side is thus seen to be overcome by the dead weight, with a sufficient margin of safety, even for a building in which the floor filling, partitions and plastering remain to be done; and anchorage becomes unnecessary.

After all, it seems that in narrow, clear span buildings, especially with the heavier fire-walls now demanded, rigid connections to the columns at all floor levels, with girders and columns designed to stand an additional vertical load, equivalent to a given wind pressure over the exposed wall, forms a good solution of the problem. It will generally be found that the moment of the wind acting through the center of pressure, is exceeded by the moment of the dead weight.

HENRY H. QUIMBY, M. Am. Soc. C. E.—The forces which towering buildings must be braced to resist are so erratic and afford such limited facilities for investigation, that we find acknowledged masters of the science of structural design differing widely in their views of them, and displaying their peculiarities of judgment in their professional practice. Probably in no other department of engineering is there such a woful lack of scientific agreement. The exigencies of business make economy a requisite in a design only second in importance to that of safety; but in designing large and important buildings, architects are not driven to the last notch of their judgment and conscience, as competition often drives bridge builders, and they should not, without good reason, take the risk of omitting efficient wind bracing.

A certain foreign engineer, in commenting on the boldness of some Americans in the construction of public works, said that our characteristic reflection seems to be, "Well, let us try it." Probably this correctly represents the disposition of some who are waiting for a disaster to occur to justify advanced methods, as the Tay Bridge failure did in England.

The subject treated of here in its application to very high buildings is, as President Cohen remarks, comparatively new, and we must base our judgment of possibilities on the observed effects of natural forces on lower and differently constructed buildings. We do know of winds that have literally and absolutely swept everything before them, and left no measure of their intensity. We know that several so-called tornadoes have traversed our eastern cities and blown down comparatively low and large masonry buildings, and we know also that no community can claim to be exempt from the danger of them. The fact that they occur at intervals of perhaps years, and are of short duration and have narrow paths, minimizes the chances of getting reliable observations, but do not justify ignoring them; and it is the duty of the architect or engineer to decide what prudence requires shall be provided for.

This paper suggests 40 pounds per square foot of exposed surface as an assumed load to secure reasonable provision against all the destructive forces referred to—natural and artificial. Mr. Just thinks that 30 pounds is a safe assumption. Others, who don't say anything, seem to provide for much less. Thirty pounds may be safe, and 40 pounds may be not safe; but the latter, while much more to the right side, can be objected to only in the line of economy, and the addition of one-third to a well-designed system of diagonal bracing, and one-ninth to the outer columns, will be a small percentage of the total cost of a building where doubtless large sums are spent for decoration alone.

Mr. Just's plan of a single system of stiff diagonals is good and avoids interference with passage doors; but in wider buildings it involves troublesome splices at intermediate columns, and it does not admit of convenient adjustment, which is desirable as a means of plumbing the successive tiers of columns and correcting the effect of unequal settlement. This can be best obtained with tension members which are convenient of attachment, though not always most economical of material. The lateral stiffness of the floors makes it unnecessary to have bracing in every bent, and permits the shift of bracing to different bents in different stories if needed for architectural purposes.

But Mr. Just, after solving this problem as it appeared in a partially completed building, by inserting an iron vertical wind truss, concludes his discussion with the opinion that the solution of the general problem is in rigid floor connections and heavier columns, presumably leaving the diagonal stresses to be cared for by the partitions. In his typical building (Fig. 6) he shows a diagonal stress in the first story of 38.5 tons produced by a wind pressure of 40 pounds. A pressure of 30 pounds, which he advocates as proper to provide for, will develop stress amounting to 29 tons. Would he concentrate such a load upon a hollow brick wall with a total net thickness of $2\frac{1}{4}$ inches

or less, as modern fireproof partitions are built? Does anybody know how much such a wall 12 feet high can safely carry?

In the seventeen-story building described in the paper, with 30 pounds wind pressure, the shear at the top of the adjoining six-story building (a support that may sometime be removed) is 39.5 tons per panel of 20 feet. As there are three interior rows of columns, this shear is resisted by four walls, each 15 feet wide and 12 feet high. The diagonal compression in each is therefore 13 tons, all of which is assumed to come upon the partition walls because of the flimsy character of the column connections (Fig. 1 in the paper). Thirteen tons make a heavy load for such walls, even when they are not, as many of these are, weakened for diagonal resistance by doorways through them.

Rigid floor connections, if they are as good as the detail shown by Mr. Milliken, will, with the ordinary partitions, furnish sufficient stiffness for buildings having a good breadth proportionate to height, and will materially assist the diagonal braces in a narrow, high structure. Mr. Milliken's claims of excellence in his design are well founded if the pintle has the requisite number of rivets; but it can scarcely be expected that shop work will be so accurate that the rivet holes will bring the lines of columns perfectly plumb, as his fourth point seems to claim.

The consideration of earthquake and other vibratory forces was presented, not with a view of fixing definite measures of force, but as affecting the wind-bracing problem in the direction of justifying larger provision, and especially as a means of showing the importance of having a building stiffened by something of more positive and lasting strength than light brick masonry. It is to be regretted that the discussion has shed no light on the value of the hollow tile walls which are so much used for partitions and bracing; but it has brought out illustrations of the destructive effect of vibrations on solid walls, and strengthened the position taken in the paper, that hollow tile is not the most efficient bracing for buildings subject to continued vibration.

Replying to Mr. Bogart, the steel skeleton type of building is new, and, so far as known, not one has yet been destroyed by wind; but ordinary buildings have been blown down presumably because they were not properly braced. A few years ago a three-story factory near my home was demolished—simply pushed over without evidence of cyclonic action—in a severe storm. It had been in use three years with light machinery. The walls were of brick, 22 inches thick in the first story and 18 inches in the second and third, with 4-inch pilasters every 12 feet. More than half of the wall space was taken up with windows. The building was 260 feet long, 52 feet wide, and 48 feet high, and a 7-foot lantern-top crowned the roof. In the middle, at one side, was a stair tower about 12 feet square, and between it and each end were closet bays projecting 4½ feet. These bays and the tower

remained standing, but the remainder of the building was leveled to the sills of the first-story windows. The second and third floors were supported by one row of wooden posts 12 feet apart, but there were no knees or partitions anywhere. Apparently, good bracing would have saved the building and prevented the loss of twenty lives.

The peculiar action of wind currents observed by Messrs. Dunham and Harris confirms the theory advanced in the paper that wind accumulates intensity by being deflected.

The efficient system of bracing in an eight-story building described by Mr. Snow, is interesting in comparison with the plan of the seven-teen-story building referred to in the paper, and shows that the subject has already received advanced treatment.

In a field of so much uncertainty, increased provision is in the nature of insurance, and, like fire insurance paid on a combustible structure that is never damaged by fire, should not be regarded as wasted, but as returned in the shape of protection.

RAINFALL, FLOW OF STREAMS, AND STORAGE.

By Desmond FitzGerald, M. Am. Soc. C. E.

Read June 8th, 1892.

WITH DISCUSSION.

The accompanying tables were prepared during the summer of 1891, for the purpose of calculating the yield of drainage areas with varying proportions of land and water surface. The results contained in this paper are intended for use in Massachusetts. They may, perhaps, be found applicable to a very much larger area.

Rainfall.—There is hardly any phenomenon about which so many misstatements are commonly made as that of rainfall. Either, "the cutting down of the forests is fast diminishing the annual precipitation" or else the latter is "increasing rapidly from turning up of the ground," and other causes. "There are no longer such snow storms as we used to have." "The rains come now altogether in the spring." "'Freshets' and 'droughts' alike come from great changes in the rainfall." These and a multitude of other fallacies are constantly met with. As a matter of fact, the annual rainfall is such a varying quantity that it is

extremely difficult to lay down general laws in regard to certain of its phases, even with the aid of a good rainfall table.

Again, the observations themselves are frequently inaccurate, as can sometimes be told at a glance. The earlier results were generally too small, because the gauges were placed too high and less care was exercised to measure all the small showers and the snow. Too often the tables issued from official sources and stamped with the approval of the Government are open to this criticism.* The periods also are generally too short to build safe theories upon; and, lastly, self-interest connected with important commercial enterprises leads to false statements.

Table No. 1 contains a compilation of seventy-four years of rainfall, by months, in the vicinity of Boston, and is now first made public. Another table, not here published, contains a record of rainfall observations 1852-91, made at Lake Cochituate, about 15 miles from Boston, and Table No. 2 gives the rainfall on the Sudbury River water-shed from 1875-90 inclusive.

Yearly Means.—The yearly means from these tables are as follows:

        Boston, seventy-four years...................47.00 inches.
        Cochituate, forty years.....................47.98   "
        Sudbury, sixteen years......................45.80   "

An examination of the yearly fluctuations from these means, taken in connection with the methods of making the observations, does not disclose any definite law of increase or decrease. If there is a secular change, it is probably too slight to be observed in a century, especially where the observations are not all taken under exactly the same conditions and those conditions such as experience has shown to be necessary. The Providence and Lowell records make the average for the year about 45 inches.

As there is a liability to underestimate rather than to overestimate the rainfall,† the writer assumes that a general average for Boston cannot be far from 48 inches, or 4 inches per month.

Maximum and Minimum Rainfall.— The largest annual rainfall recorded in Table No. 1 occurred in 1863, 67.72 inches, and the smallest in 1822, 27.20 inches. If these figures are correct, they show how great

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* The Signal Service observations of rainfall made on the tops of high buildings are untrustworthy.

† Largely from placing the gauge too high above the surface of the ground.

the range is. They cannot be far from the truth, because in 1883 the
record of 32.78 inches on the Sudbury is corroborated by the record of
many gauges, and the rainfall tables of Lowell, Providence, Waltham
and other places all point to a minimum of about 30 inches. The
writer has ascertained by an examination of the original records that
the rainfall recorded at Waltham, of 26.9 inches, in 1846, included ten
months only, and that the record at Lowell of 28.46 inches in 1825
contained nine months only. Such facts as these are sufficient to
make us exceedingly cautious in regard to records. In a general way
it is safe to say that the yearly rainfall varies from 30 to 60 inches.
The minimum monthly rainfall was 0.23 inches in September, 1884, and
the maximum properly recorded in any one month is probably not far
from 12 inches.

Monthly Means.—The following table shows the monthly means:

	BOSTON, 1818-91.	COCHITUATE. 1852-91.	SUDBURY, 1875-90.
January...............	3.98	3.88	4.18
February.............	3.78	3.62	4.06
March................	4.36	4.25	4.58
April................	4.06	3.97	3.32
May..................	3.79	3.87	3.20
June.................	3.27	3.31	2.99
July.................	3.71	4.23	3.78
August..............	4.39	4.94	4.23
September...........	3.55	3.59	3.23
October.............	3.84	4.29	4.41
November...........	4.31	4.44	4.11
December...........	3.96	3.59	3.71
	47.00	47.98	45.80

The progress of the monthly fluctuations can be seen in the diagram
on the following page (Fig. 1).

There is a strong similarity in the profiles, too decided to be the
subject of chance. The Providence rainfall (1832–91) has been added
in a series of small circles.* They correspond to the general form of
the other lines.

Whatever doubt we may have in regard to individual observations,
the general accuracy of the average monthly distribution must be con-
ceded. Longer observations may change the maximum and minimum
points, but the present weight of evidence seems to favor March,
August and November for maxima, and June and September for minima.

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* The Providence rain gauge previous to 1876 was 7 feet above the ground, and must
have given too small a result.
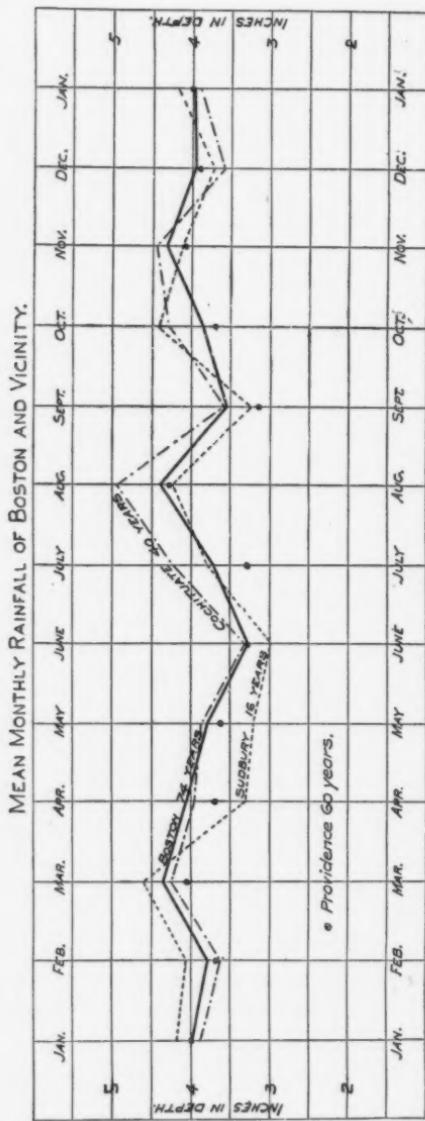
Fig. 1.
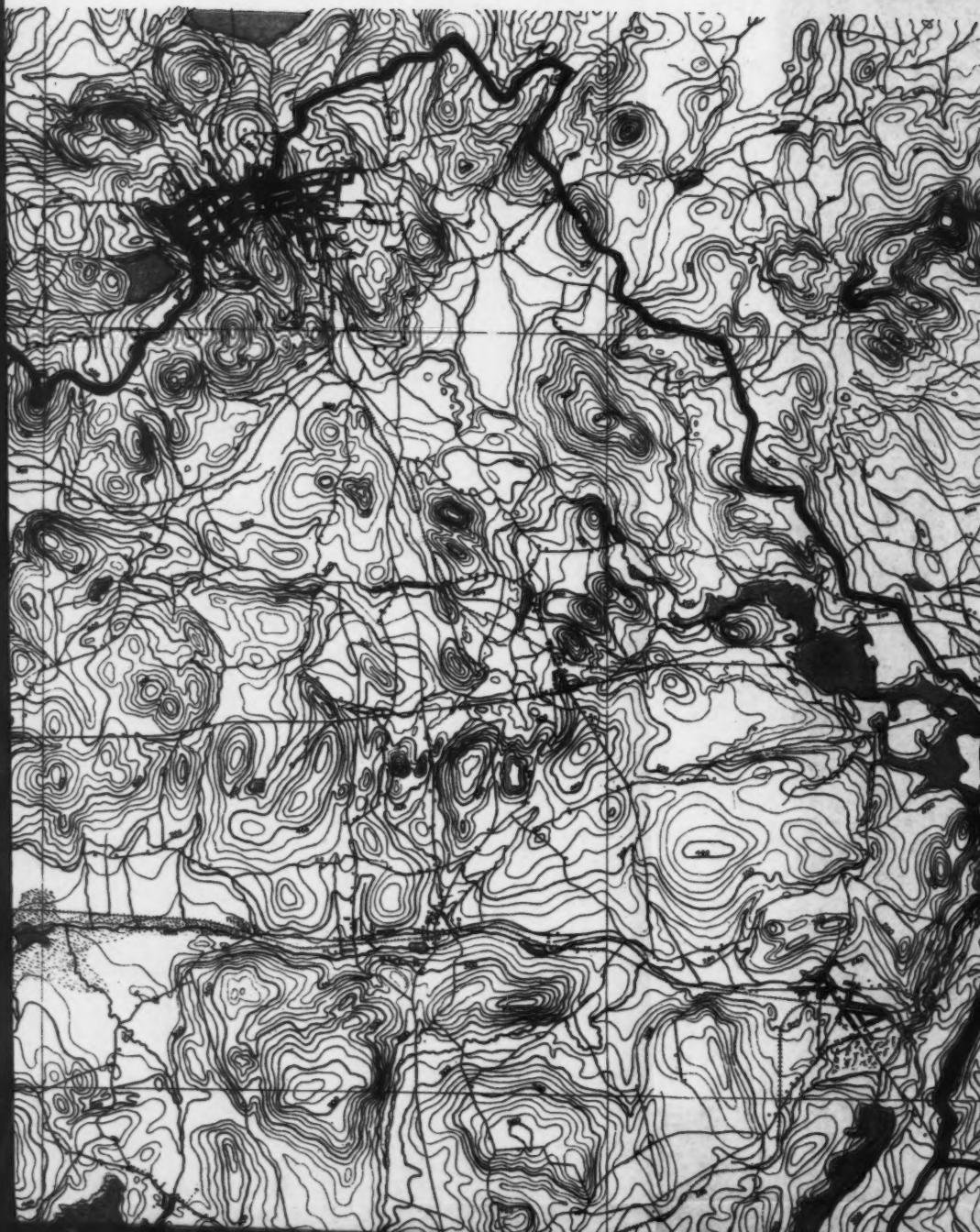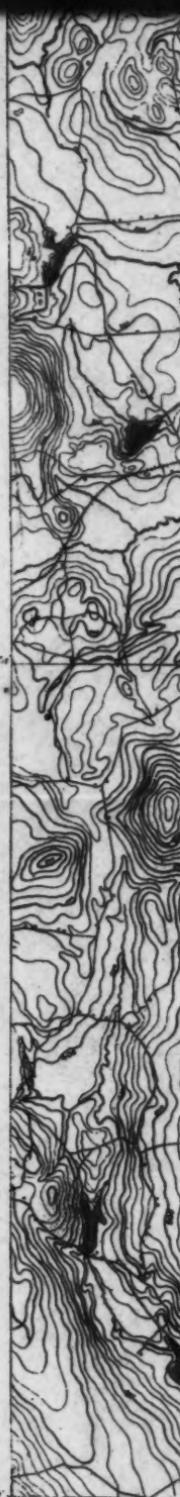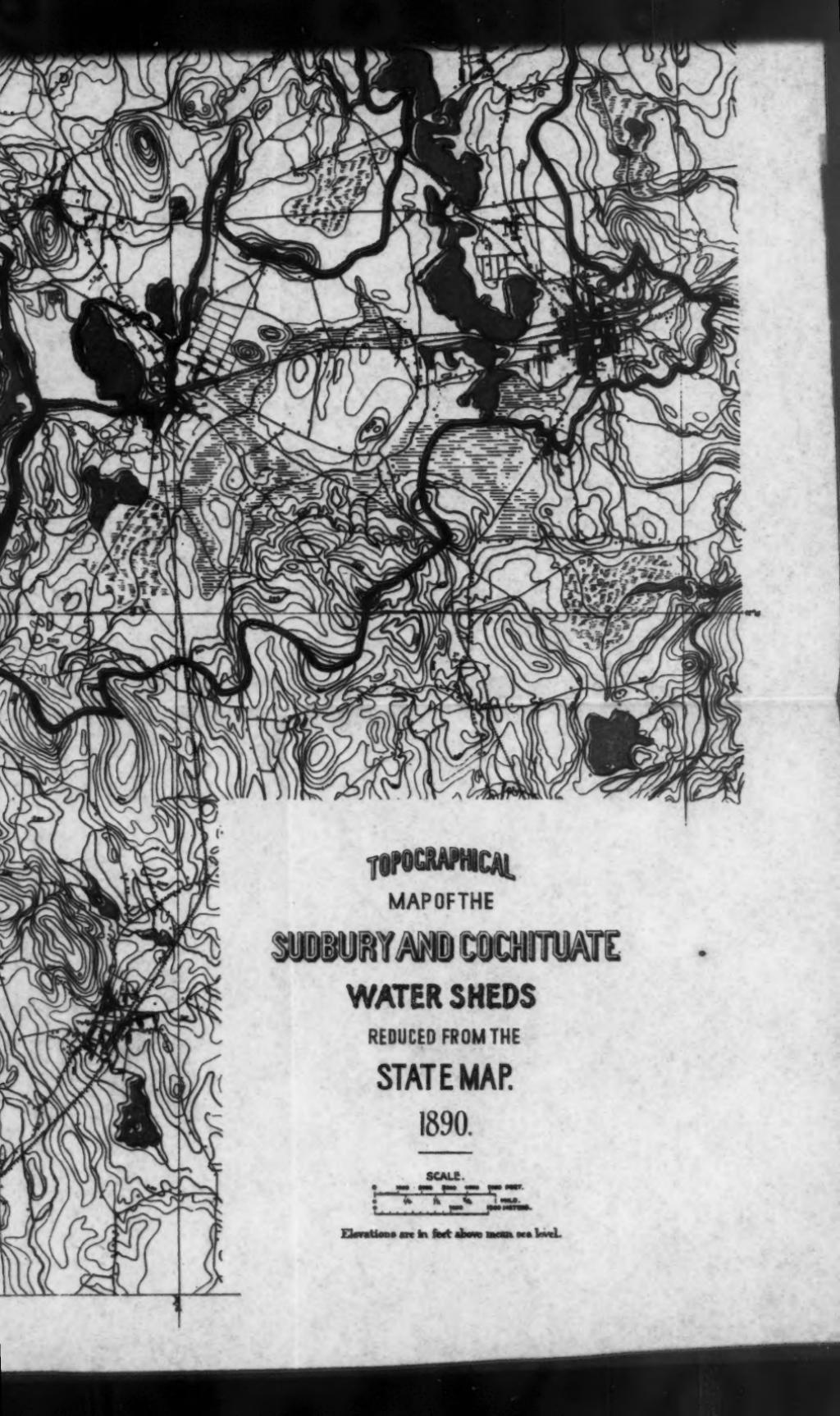
TOPOGRAPHICAL

MAP OF THE

SUDBURY AND COCHITUATE

WATER SHEDS

REDUCED FROM THE

STATE MAP.

1890.

SCALE.
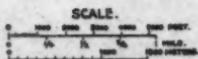
Elevations are in feet above mean sea level.

Seasonal Distribution.—The seasonal distribution is as follows:

	BOSTON.	COCHITUATE.	SUDBURY.
Spring	12.21	12.09	11.10
Summer	11.37	12.49	11.00
Autumn	11.70	12.31	11.75
Winter	11.72	11.09	11.95

From these figures it is obvious that the rainfall is evenly distributed through the seasons of the year.

Evaporation.—The total flow of a stream must equal the rainfall, less the evaporation and other losses. As the latter are generally insignificant, the difference between the rainfall and the rainfall collected, is the total evaporation from the water-shed supplying the stream. The average of a number of years shows us that from 45 to 50 per cent. of the rainfall flows away in the stream; and if the average rainfall is 48 inches, then about 24 inches are evaporated yearly from the ground and other surfaces ordinarily found on a water-shed. The evaporation from a water surface is greater than that from the ground. Table No. 5 is an attempt to represent the monthly evaporation from a water surface during the period embraced in the other tables. The data upon which the table is founded are taken from a paper on " Evaporation," published in the *Transactions* of this Society in 1886 (Vol. XV, p. 581), but some observations made since the paper was published have been added. It appears from the table that the mean evaporation from a water surface in Boston is 39.2 inches, or about 82 per cent. of the mean rainfall, although it must be remembered that there is no connection between rainfall and evaporation. The diagram on page 258 (Fig. 2) is a new diagram of mean evaporation, which contains additional data on that already published.

Description of Water-Sheds.—The topographical map (Plate XLV) accompanying this paper gives an idea of the nature of the Sudbury and Cochituate water-sheds, but a few words of description seem necessary.

The Sudbury River water-shed has an area of 75.199 square miles; the Mystic, 26.9 square miles; and the Cochituate, 18.87 square miles. They together form the sources of Boston's water supply. The Sudbury is hilly, with steep slopes. There are, however, some large swamps within its borders. The Cochituate, although adjoining

the Sudbury, is entirely dissimilar.   The slopes are flat and sandy.
Its surface is mostly modified drift, while the Sudbury is largely com-
posed of unmodified drift.   The Mystic water-shed lies to the north of
Boston, and about 30 miles distant from the other two sources which
are to the west of the city.   Its surface is steeper than the Cochituate,
and not as steep as the Sudbury.

MEAN MONTHLY EVAPORATION CURVE.
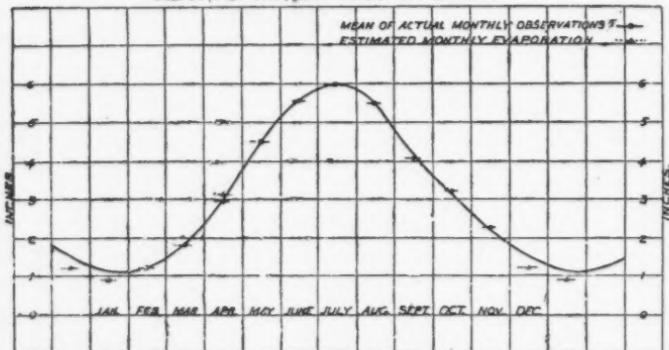
FIG. 2.

Flow of Streams.—However uniformly the rainfall is distributed, on
the average, throughout the year, the effect of the excessive evapora-
tion during the summer and of the frozen ground with accumulations
of snow from month to month in the winter, is to produce a most
irregular flow in the streams.   During seven months of the year, from
November to May inclusive, the streams have a large flow, and during
five months, from June to October, a small flow ; but it is in February,
March and April that we must look for the very large yields.

Table No. 15 contains the yields of the three water-sheds for various
periods and combined in several ways.   The results are different in
some particulars, but in general agree sufficiently well to form the basis
for an instructive investigation.   The widest variation that is found in
the tables, exists between the Sudbury and Cochituate collections.   On
the average, the latter collects 12 per cent. less water than the Sudbury.
It is probably true that the Sudbury water-shed gives a somewhat
larger yield than the average water-shed.   It collects much more in
the spring than the other two water-sheds, and rather less in the
summer.   It is impossible in the limits of this paper to go into an

extended discussion of the causes leading to these differences in collection, but the writer has thought that a better average, or typical collection, would be obtained by uniting the results of all three.

Monthly Yield.—In the diagram on page 260 (Fig. 3) the average monthly yields of the three water-sheds have been plotted for the period 1878–90, inclusive. The heavy line is a mean of the three, and for convenience the values are here repeated with their equivalents in cubic feet per second.

YIELD OF A TYPICAL NEW ENGLAND WATER-SHED PER SQUARE MILE.

	GALLONS.	CUBIC FEET PER SECOND.
January..........................	37 387 000	1.866
February.........................	55 056 000	3.042
March............................	71 226 000	3.555
April............................	49 107 000	2.533
May..............................	30 406 000	1.518
June.............................	14 975 000	0.772
July.............................	7 491 000	0.374
August...........................	11 399 000	0.569
September	10 242 000	0.528
October..........................	16 797 000	0.838
November.........................	24 787 000	1.278
December.........................	34 128 000	1.703
Total and Mean............	363 001 000	1.539

It will be convenient for many purposes, especially where questions connected with the use of water for purposes of power are concerned, to use these monthly results in the order of their magnitude as follows:

AVERAGE MONTHLY YIELD PER SQUARE MILE IN ORDER OF MAGNITUDE.

Cubic Feet per Second.

July.............................	0.374
September	0.528
August..	0.569
June.............................	0.772
October...	0.838
November.......................................	1.278
May	1.518
December..	1.703
January...	1.866
April...	2.533
February..	3.042
March...	3.555
Mean...............................	1.539

Fig. 4 contains the above quantities in graphical form.
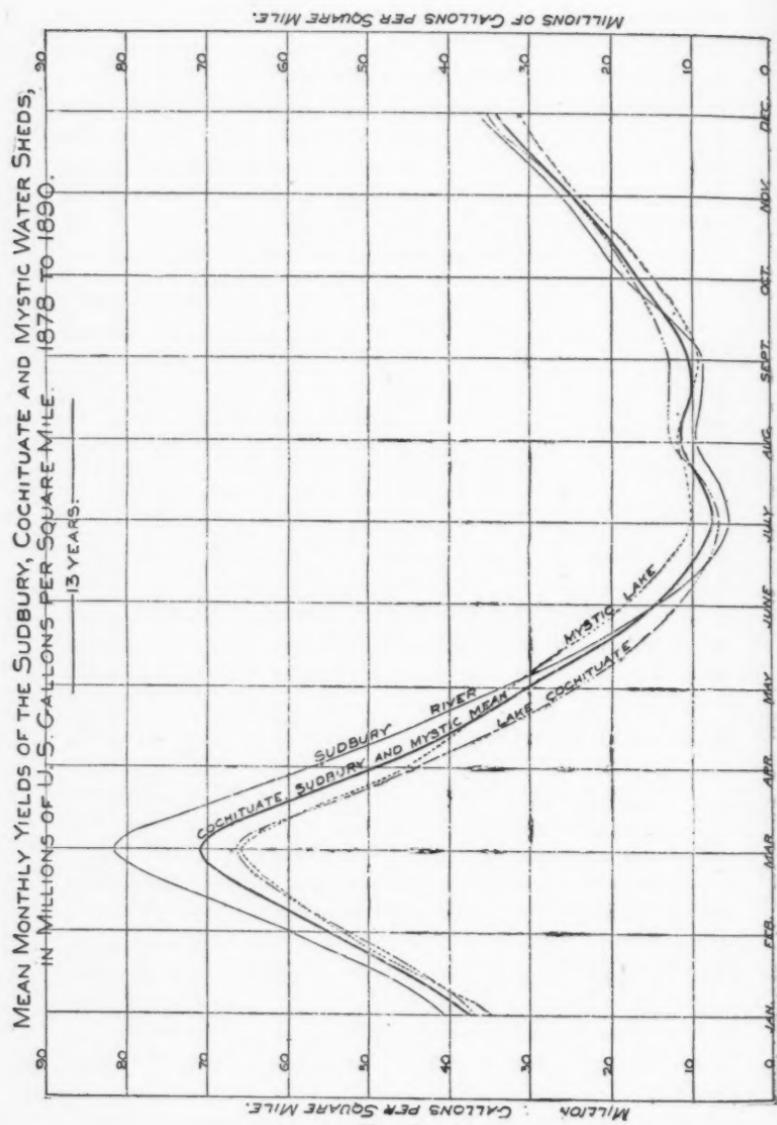
FIG. 3.

The average yield of the Sudbury River water-shed alone for sixteen years is 1.669 cubic feet per second, which corresponds very closely to the Croton water-shed mean of 1.626.

The month of May represents the mean monthly flow for the entire year.

Ordinary Flow of a Stream.—The ordinary flow of a stream certainly does not mean the average flow, for one or two heavy freshets occurring in a few days in the spring of the year have a great influence on the
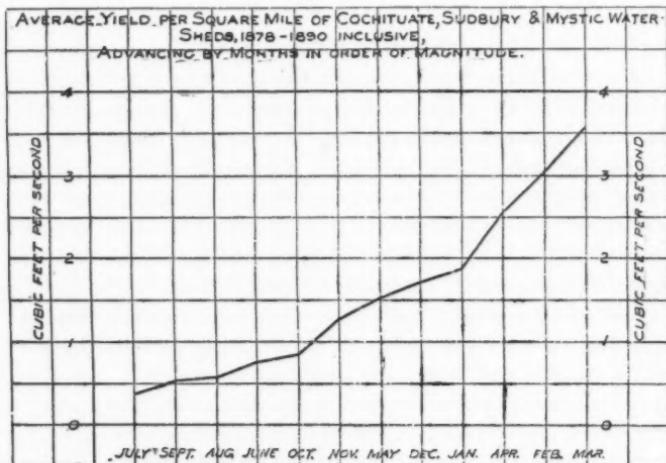
FIG. 4.

average without affecting in any way the ordinary run of water. The large flows above the average, occurring at the summit of the diagram (Fig. 4), in the months of February, March and April, should evidently be excluded. The flow in July is too insignificant to be considered. If we take the months above the average, at the average rate, and add to these the months below the average, excluding the month at the bottom of the list, and divide by the whole year, we shall, in the opinion of the writer, get a fair ordinary flow of the stream.

Applying this rule to the results already obtained we have :

<div align="center">Gallons.</div>

September.........	10 242 000
August............	11 399 000
June..............	14 975 000
October...........	16 797 000
November.........	24 787 000
May..............	30 406 000
Five months of mean flow,	149 178 000

257 784 000 ÷ 365 = 706 000 gallons per day, or about 1.1 cubic feet per second per square mile.

Maximum Yield.—On the average, March is the month of maximum yield and its flow is about two and one-third times the mean monthly flow and about one-fifth the entire flow for the year. In March, 1877, the Sudbury water-shed (Table No. 8) yielded 149 222 000 gallons per square mile, or 7.448 cubic feet per second per square mile ; about one hundred times the minimum monthly yield. This flow averaged 4 807 000 gallons daily per square mile for thirty-one days. It is the largest flow recorded for a month in the sixteen years.

Freshets.—The greatest freshet occurring on the Sudbury water-shed took place February 10th–13th, 1886, and a full description of its effects can be found in the *Transactions* of the Society for September, 1891. The maximum yield for twenty-four hours was equal to 1.54 inches in depth upon the surface, or 26 763 260 gallons per square mile, or 41.4 cubic feet per second per square mile; and the maximum rate of yield was equal to 1.646 in depth on the water-shed in twenty-four hours, or 44.2 cubic feet per second per square mile. On March 26th, 1876, a freshet giving nearly the same yield occurred, so that it is not probable that this amount of rainfall collected in twenty-four hours is very unusual.

Gaugings on a small affluent of the Sudbury, embracing 6.434 square miles of surface, showed a similar flow per square mile to that in the main stream. The maximum rate for twenty-four hours in the small affluent was equal to 1.801 inches of depth on its drainage area. Although these freshets were considered very disastrous in Massachusetts, they cannot be considered large when compared with what has been observed in neighboring water-sheds. Freshets of 3 and 4 inches

collected in twenty-four hours are within the experience of many hydraulic engineers, and it certainly would not be safe in designing dams to provide for less than 6 inches collected in twenty-four hours and flowing continuously, or 104 272 440 gallons per square mile per twenty-four hours, or 161.3 cubic feet per second. These remarks are of course applicable only to an ordinary New England water-shed not covered with houses and streets. The water-works engineer who is constantly designing waste weirs, dams, reservoirs, etc., may find it convenient to bear in mind that one square mile of land surface yields approximately 1.5 cubic feet per second throughout the year and that the maximum freshet flow may be one hundred times this amount or 150 cubic feet per second. In millions of gallons these become one million and one hundred millions respectively.

Minimum Yield.—The minimum daily yield of a stream is a dangerous subject for figures. There are so many conditions tending to affect the daily flow, even on the average water-shed, that it may often be something that man controls by his use of mill dams, storage basins, etc. Taking a period of a month, the Sudbury has shown in September, 1884 (Table No. 8), a yield as small as 1 318 000 gallons per square mile, or 43 930 gallons daily per square mile, equivalent to 0.068 cubic feet per second per square mile. The water surface during this month was 0.0303 of the total area.

Clemens Herschel, M. Am. Soc. C. E., reported to this Society in July, 1881,* that the Connecticut River, with 3 287 square miles of drainage area, had discharged as low as 0.306 of a cubic foot per second per square mile. Other large rivers in the country have shown a lower yield than this.

The discharge of a small water-shed, say, 2 or 3 square miles in area, in a protracted drought may be practically nothing. The geological formations on a water-shed have an important bearing on the minimum yield. If there are large plains of loose gravel or sand which hold water, and whose water tables are drawn down in the summer, the minimum flow is larger than where the same areas are occupied by unmodified drift.

Average Daily Yield.—It is apparent from an inspection of the tables that the average daily yield per square mile of an average water-shed is about 1 000 000 gallons, or 1.5472 cubic feet per second.

* Vol. X, *Transactions*, page 238.

Percentages of Rainfall Collected.—Percentages are almost as danger-
ous to deal with as the minimum flow.   In many ways percentages are
instructive.   They bring home forcibly to the mind the relation existing
between the rainfall and the flow in the streams at different seasons of
the year, but their results must be interpreted with a knowledge of the
prevailing conditions.

Table No. 4 contains the monthly percentages of rainfall collected
on the Sudbury for sixteen years.   To the general monthly averages
of this table, the monthly averages in the Cochituate water-shed from
1863–91, inclusive, have been added in the following table for purposes
of comparison.

<div align="center">MEAN PERCENTAGES OF RAINFALL COLLECTED.</div>

	Sudbury, 1875–90.	Cochituate, 1863–91.
	Per Cent.	Per Cent.
January	49.1	53.1
February	78.2	71.9
March	109.6	84.6
April	109.1	83.8
May	62.3	47.9
June	29.1	27.9
July	8.9	13.1
August	13.0	17.7
September	14.2	23.5
October	23.1	23.6
November	39.5	34.0
December	52.5	49.9
Mean	49.5	43.8

The percentages exceeding 100 are caused by accumulations of rain-
fall, generally in the form of snow, from one month to another, passing
off finally in a different month from that in which it fell.   In consider-
ing monthly collections, it must not be forgotten that the amount of
rain falling in the previous month has a large influence on the col-
lections of the month following, especially in porous water-sheds.   A
knowledge of this fact will often explain what seem to be anomalous
results.

The yearly percentages of rainfall collected on the Sudbury have
varied from 31.9 per cent. in 1880 to 62.2 in 1888, and on the Cochituate
from 25.7 per cent. in 1866 to 69.1 per cent. in 1891.   The mean for six-
teen years on the Sudbury is 49.5 per cent., and for twenty-nine years
on the Cochituate 43.8 per cent.   The percentages depend upon the

PLATE XLVI.

750,000

900,000

850,000

900,000 GALS PER

520   540   56

10
9
8
7
6
5
4
3
2
1

250   280   300   320   340   360   380   400   420   440   460   480   500
MILLIONS OF GALLONS STORAGE
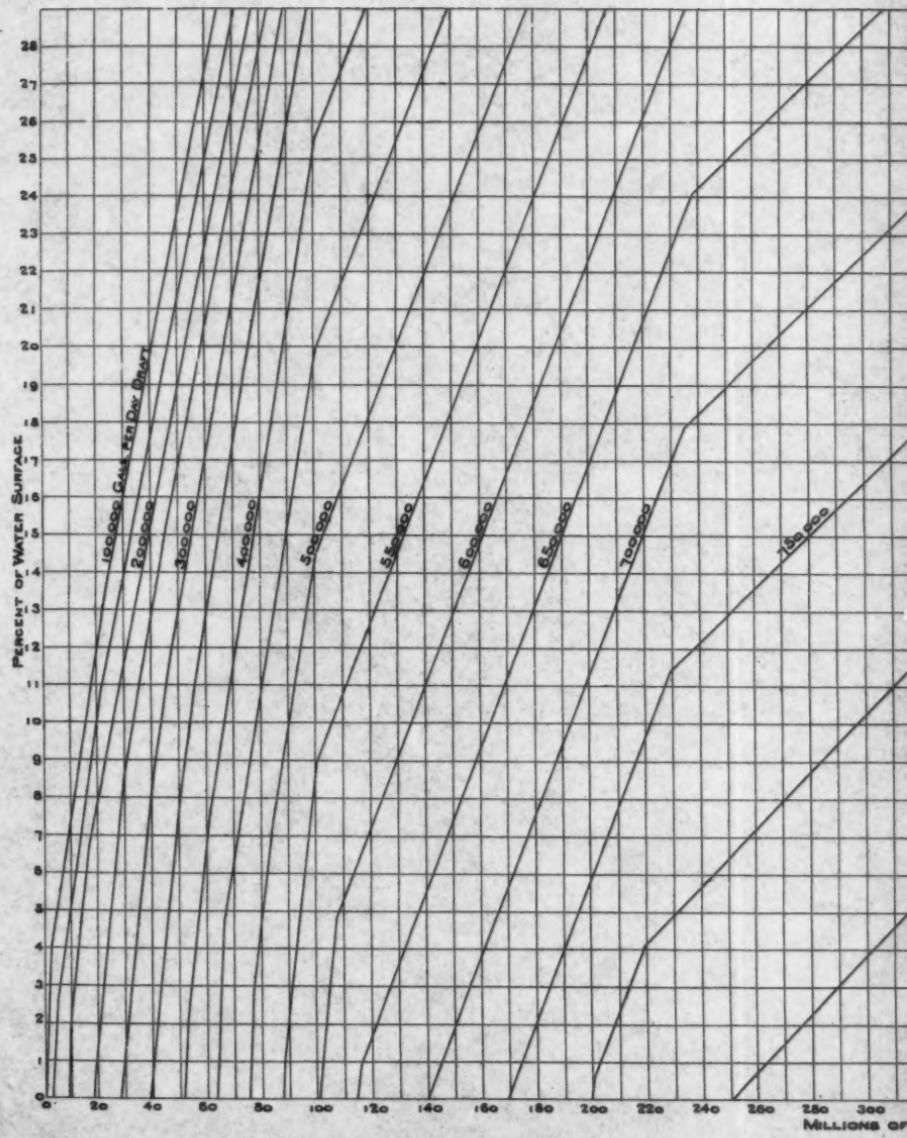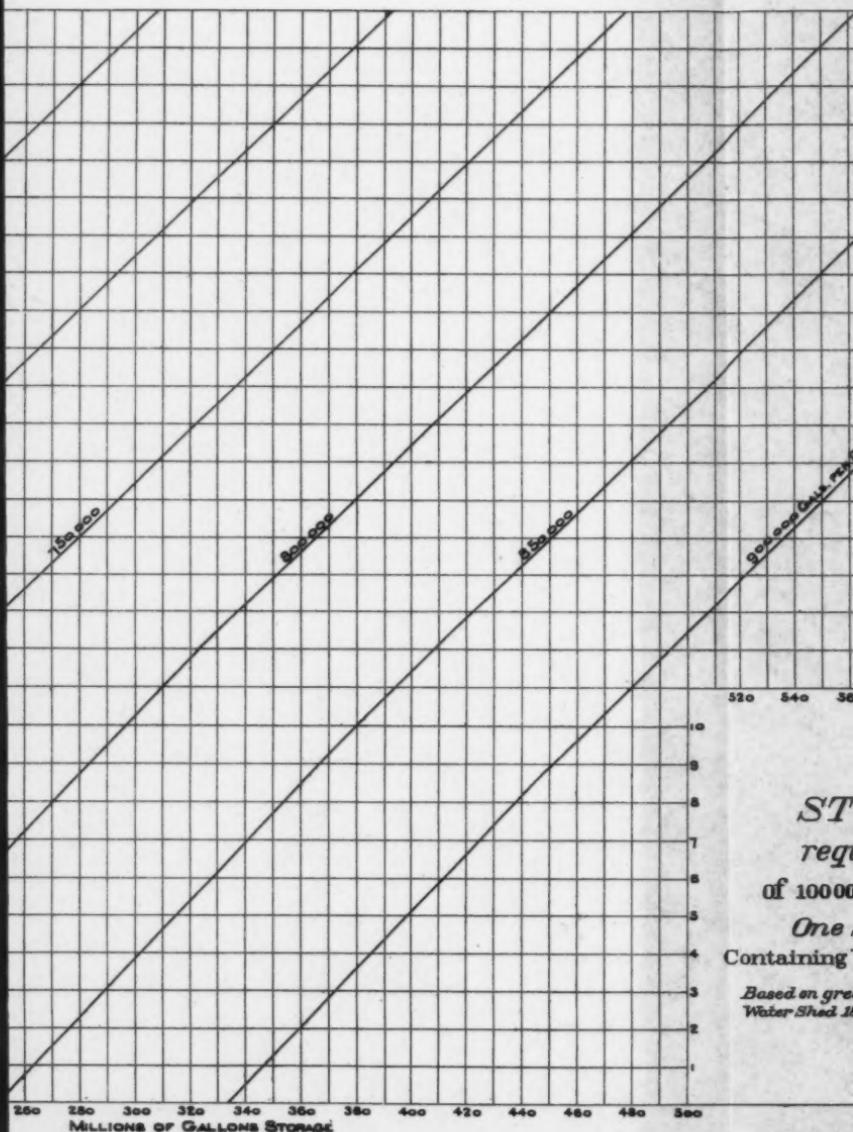
ST
req
of 10000
One
Containing
Based on gre
Water Shed 1

340   360   380   500   620   640   660
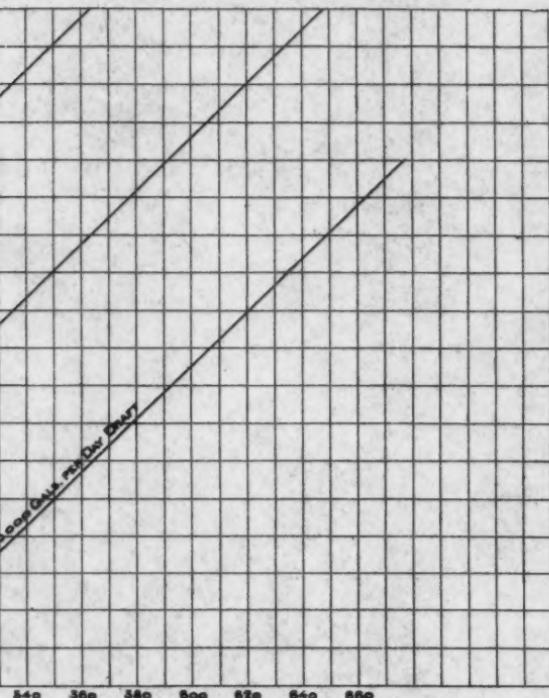
STORAGE CAPACITY

required to Sustain Drafts

of 100000-900000 Gallons Daily from

One Square Mile of Water Shed

taining Various Percentages of Water Surface

ased on greatest droughts occuring on the Sudbury River
ter Shed 1875-1890.

May 1892.

DESMOND FITZGERALD.

distribution of the rainfall throughout the year. A heavy summer rainfall and a light winter rainfall mean a small percentage of collection, and, conversely, a light summer and a heavy winter rainfall mean a large percentage of collection, so that the total rainfall for the year is but a partial index to the yield of a water-shed.

If, on the average, 11 per cent. only of the rainfall in July is collected, it becomes evident that the amount of the rainfall in that month, speaking within ordinary limits, is of but little consequence for most purposes connected with water supply; and, still further, it may be said that for systems which depend upon storage, it is not the summer droughts which are to be dreaded, but the winter and spring droughts; for it is the flow in these months upon which we depend to fill the reservoirs. In July, 1876, the percentage of rainfall on the Sudbury fell to 3.6.

Storage.—Owing to the great variations in flow, as already shown, the question of storage becomes of great importance in schemes which look to developments of available supplies from water-sheds. In a report made by the writer in 1887 on "The Available Capacity of the Sudbury River and Lake Cochituate Water-Shed in Time of Drought," a method of showing graphically the storage required in the periods of deficiency from 1875 to 1887 was given, in the form of a mass curve after Rippl.*

This curve is found by plotting the differences between the yields and the daily drafts. By this method the storage required for the given daily draft in the periods of deficiency is easily found.

By means of Tables Nos. 11 and 13 (pages 281 and 283), which give the yield of the Sudbury water-shed for sixteen years, by months, reduced to 1 square mile of land surface and 1 square mile of water surface, it will be possible to find the yield of any water-shed, whatever its ratios of land and water surface.

Having found the yield, it will then be an easy matter to ascertain the extent of the periods of deficiency for any given draft, or the periods when the yield is less than the draft. To save the trouble of these computations, the table (page 267) has been prepared, showing at once the maximum storage required to tide over the droughts between 1875–90, and for different daily drafts from 100 000 to 900 000 gallons, and ap-

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* W. Rippl. "The Capacity of Storage Reservoirs for Water Supply." Min. Proc. Inst. C. E., Vol. 71, p. 270.

plicable to different percentages of water surfaces found upon the given water-shed. The table is reduced to the unit of one square mile, so that a simple multiplication will give the required storage upon any given number of square miles.

As the greatest period of deficiency or the most prolonged and severest drought is the one on which this table is based, and must ever be the one to which the cautious engineer turns for instruction, it may be interesting to remember that this period from 1875 to 1890, inclusive, contains two years of most remarkable drought, following each other closely. These were the years 1880 and 1883; in fact, the period of yield from 1879 to 1884 forms a crucial test or measure of what may be expected in the future. The value of exact measurements during this period will be the better appreciated when we take into consideration that the rainfall records for sixty years, 1830-90, give no evidence of any more severe period of drought.

Of the future certainly we know nothing; but, judging by the past, we may feel assured that we have done all that a reasonable care demands, if our works are proportioned to the maximum drought occurring in so long a period as sixty years.

The following is an example of the use of the table. Suppose we have a water-shed of 40 square miles which contains 10 per cent. of water surface, and we wish to draw daily 500 000 gallons. The question is how much storage must be provided. Look in the left hand column for 500 000, and follow this line to the column headed 10 per cent.; here, we find the figures 90 550 000 gallons storage per square mile, which, multiplied by 40, gives 3 622 000 000 gallons—the required storage.

The diagram Plate XLVI, which is a plot of the table under discussion, will facilitate the taking out of quantities for intermediate percentages not given in the table. The horizontal lines are the ratios of water surface. The perpendiculars dropped from the intersections of these horizontal lines with the various lines of draft will give, at the bottom of the diagram, the required storage in millions of gallons.

When storage reservoirs are distributed on different portions of a water-shed, it is important to consider them with reference to their positions, the extent of their individual drainage areas, and their capacities for yield in connection with the other portions of the water-shed; and these studies afford some of the most interesting problems for the water-works expert. It is believed that the data collected in this paper will give the means for solving these problems.

TABLE SHOWING STORAGE CAPACITY REQUIRED TO SUSTAIN A CONSTANT DAILY DRAFT FROM ONE SQUARE MILE CONTAINING VARIOUS PERCENTAGES OF WATER SURFACE.

Based on Sudbury River Water-Shed.—Sixteen Years.—U. S. Gallons.

Constant Daily Draft.	0 per cent.	2 per cent.	4 per cent.	6 per cent.	8 per cent.	10 per cent.	15 per cent.	20 per cent.	25 per cent.
100 000	314 000	1 289 000	2 656 000	6 973 000	10 992 000	15 012 000	26 883 000	40 224 000	53 565 000
150 000	3 006 000	4 711 000	7 552 000	11 573 000	15 592 000	19 642 000	32 963 000	46 324 000	59 665 000
200 000	8 797 000	9 937 000	12 802 000	15 666 000	20 427 000	25 742 000	39 083 000	52 424 000	65 765 000
250 000	17 997 000	20 637 000	23 502 000	26 366 000	29 230 000	38 338 000	45 449 000	58 524 000	71 865 000
300 000	28 473 000	31 337 000	34 202 000	37 066 000	39 930 000	43 437 000	54 599 000	66 702 000	78 807 000
350 000	39 173 000	42 037 000	44 902 000	47 766 000	50 630 000	54 137 000	64 812 000	75 832 000	87 967 000
400 000	51 303 000	52 788 000	55 602 000	58 466 000	61 643 000	66 650 000	77 062 000	88 076 000	99 089 000
450 000	63 553 000	65 038 000	66 925 000	69 458 000	73 803 000	78 300 000	89 312 000	100 970 000	127 412 000
500 000	75 803 000	77 288 000	79 105 000	82 131 000	86 143 000	90 550 000	103 474 000	129 920 000	156 362 000
550 000	88 083 000	89 877 000	92 905 000	95 381 000	98 958 000	106 967 000	132 424 000	158 870 000	185 312 000
600 000	100 651 000	103 677 000	106 705 000	113 781 000	124 357 000	134 937 000	161 374 000	187 820 000	214 262 000
650 000	114 451 000	121 577 000	132 164 000	142 731 000	153 307 000	163 887 000	190 324 000	216 770 000	250 744 000
700 000	139 980 000	150 527 000	161 104 000	171 681 000	182 257 000	192 837 000	219 274 000	265 546 000	336 044 000
750 000	168 900 000	179 477 000	190 054 000	200 631 000	211 207 000	221 787 000	280 343 000	350 846 000	421 344 000
800 000	199 106 000	208 427 000	219 004 000	244 445 000	270 452 000	297 460 000	365 643 000	436 146 000	506 644 000
850 000	250 328 000	276 275 000	302 240 000	328 195 000	354 202 000	380 557 000	450 943 000	521 446 000	591 944 000
900 000	334 078 000	360 025 000	385 990 000	411 945 000	437 952 000	465 837 000	536 243 000	606 746 000	677 244 000

In a valuable report by Mr. Frederic P. Stearns, M. Am. Soc. C. E., to the Massachusetts State Board of Health, on "The Selection of Sources of Water Supply," the author has given a series of diagrams illustrating the fluctuations of a reservoir caused by different daily drafts. These diagrams show that, no matter what the extent of the storage may be, the effect of drafts beyond, say, 600 000 gallons daily per square mile, is to keep a storage reservoir below high water for several years.

The importance of this warning cannot be too strongly impressed upon the engineer who is designing systems of storage. The following table, calculated for 0, 10 and 25 per cent. water surfaces, shows the length of time that a reservoir would be below high water during the sixteen years we are considering, and under various drafts.

PERIODS DURING WHICH ANY RESERVOIR WILL BE BELOW HIGH WATER.

Daily draft per square mile.	PERCENTAGE OF WATER ON WATER-SHED.		
	0 per cent. Covered Reservoir.	10 per cent.	25 per cent.
100 000 gals.........	1½ months.	6½ months.	7½ months.
150 000 "	3½ "	7½ "	8½ "
200 000 "	7½ "	7½ "	8½ "
250 000 "	7½ "	8½ "	8½ "
300 000 "	8½ "	8½ "	8½ "
350 000 "	9½ "	9½ "	9½ "
400 000 "	9½ "	10½ "	10½ "
450 000 "	9½ "	10½ "	1 year, 9½ "
500 000 "	9½ "	10½ "	1 " 9½ "
550 000 "	9½ "	1 year, 9½ "	1 " 10½ "
600 000 "	10½ "	1 " 9½ "	1 " 10½ "
650 000 "	10½ "	1 " 9½ "	7 " 8½ "
700 000 "	1 year, 9½ "	1 " 10½ "	8 " 10½ "
750 000 "	1 " 9½ "	1 " 11½ "	9 " 7½ "
800 000 "	4 " 10½ "	7 " 8½ "	10 " 5½ "
850 000 "	6 " 8½ "	8 " 11½ "	11 " 9½ "
900 000 "	7 " 9½ "	9 " 8½ "	Probably about 13 yrs., 9 mos.

As it is undesirable to keep the water below high water for more than two years in succession, it will be seen that, no matter what the extent of the storage may be, it is impracticable to secure more than about 750 000 gallons daily from 1 square mile of water-shed containing 10 per cent. of water surface. As there are circumstances which permit of a different method of managing a storage basin and, where the bad influences of keeping the water low for several years are immaterial, the table for storage is carried to 900 000 gallons daily draft. With this draft a basin on the Sudbury River water-shed would have been one hundred and six and a half months or nearly nine

years, without filling to its high water-line during the period 1875–90, and a heavy growth of vegetation would undoubtedly have sprung up on the exposed margins during this long interval.

To secure this draft a storage of 377 800 000 gallons per square mile becomes necessary, which, if provided, would change the percentage of water surface from 3½ to 12 per cent., reckoning depths usually found, and requiring an increase of another hundred millions of gallons to the storage, which in turn requires another correction for increase of water surface.

The writer has had an opportunity to apply the calculated yields from various percentages of land and water surface to three water-sheds—one with no water surface, another with 5 per cent. of water surface, and the third with 25 per cent. of water surface. Daily gaugings of flow were made for sixteen months, with the results shown in the following table :

THREE WATER-SHEDS WITH VARYING PERCENTAGES OF WATER SURFACE.
Ratios of Calculated Yields to Weir Measurements.

	0 Per Cent.	5 Per Cent.	25 Per Cent.
1890.—February	0.957	0.934	0.958
March	0.925	1.031	1.005
April	1.056	0.965	0.864
May	0.945	0.865	0.859
June	1.297	1.029	0.634
July	13.391	0.713	1.742
August	4 320	0.548	−0.277
September	1.041	0.885	1.045
October	0 850	1.110	1.164
November	1.201	1.343	0.982
December	0.950	0.909	0.908
1891.—January	1.034	0.983	1.010
February	1.070	1.009	0.996
March	1.012	1.141	1.028
April	1.060	1.095	1.018
May	1.024	0.821	0.578
	1.014	1.023	0.977

With the exception of the 0 per cent., these ratios are thought to be satisfactory as showing the general accuracy of the data on which the yields for different percentages of water surface are based. A large swamp upon the water-shed, with no water surface, probably caused the failure in the case of the 0 per cent., which, however, is somewhat magnified in the table by the fact that the yield was small, though the ratios are large. The writer is inclined to caution engineers against the use of the table for 0 per cent. of water surface on account of the difficulty of finding areas of any great extent free from ponds, swamps or marshes. It is doubtful if it is advisable to use anything under 2 per cent.

TABLE No. 1.

BOSTON RAINFALL—74 YEARS.

1818–23. Dr. Enoch Hale. Carefully collated from original records. Snow melted and measured.

1823–66. Jonathan P. Hall. From 1823 to 1856 taken from Am. Acad. Arts and Sciences Memoirs Vol. VI, n. s., pp. 229, 308.*

1866–85. Supt. of Sewers.†   1885–1891. Boston Water-Works.

Year.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	For the Year.
1818..	2.64	3.49	4.05	6.15	5.96	3.47	4.08	0.46	7.81	2.11	1.91	0.86	42.99
1819..	0.70	2.27	6.21	3.74	3.06	3.56	2.02	4.38	5.29	1.40	1.22	1.63	35.48
1820..	3.12	4.25	4.90	0.45	5.08	3.42	4.19	5.15	2.43	5.39	3.00	2.80	44.18
1821..	1.41	4.42	2.53	4.90	4.84	2.79	2.35	1.58	4.73	2.29	3.12	1.93	36.89
1822..	1.29	2.34	2.02	2.99	0.78	3.54	4.32	1.20	2.15	2.53	2.58	1.46	27.20
1823..	3.00	4.57	7.72	2.21	6.40	0.93	5.74	1.98	1.95	3.95	1.92	6.93	47.30
1824..	3.95	5.99	1.81	4.72	1.43	1.60	0.88	3.68	6.43	1.01	1.72	2.80	36.02
1825..	2.79	3.43	4.70	0.37	1.36	4.77	1.24	5.62	2.66	3.21	0.81	4.38	35.34
1826..	2.55	1.48	3.81	1.50	0.25	3.85	2.90	12.10	3 03	3.80	2.31	3.56	41.14
1827..	3.92	2.97	2.51	4.75	5.34	2.56	2.59	4.88	4.81	5.28	5.71	3.59	48.91
1828..	2.15	2.79	1.84	2.00	4.67	1.59	4.58	0.37	3.82	2.79	5.55	0.26	32.41
1829..	4.93	5.62	4.30	3.45	2.71	1.64	6.98	4.95	2·62	1.65	5.74	2.26	46.85
1830..	2.36	1.63	3.51	1.21	3.93	3.46	4.90	2.64	5.65	2.38	5.32	5.96	42.95
1831..	4.44	3.68	3.07	6.97	3.65	4.32	5.53	5.57	3.83	4.42	3.20	2.93	51.61
1832..	4.47	3 74	2.65	5.56	7.27	0.50	3.41	6.14	2.07	2.46	3.57	4.85	46.69
1833..	2.96	2.53	2.71	2.30	1.03	3.23	2.01	0.82	2.88	6.00	5.53	5.86	37.86
1834..	1.39	1.13	0.96	2.93	6.33	3.09	7.71	2 47	3.71	4.62	2.90	2.36	39.60
1835..	3.25	1.37	4.27	4.54	2.07	2.74	9.07	2.89	1.31	1.87	2.08	2.40	37.86
1836..	8.84	3.57	2.90	1.58	1.85	4.33	2.12	1.53	0.54	4.04	5.43	4.13	40.86
1837..	4.10	4.14	3.02	3.07	5.79	2.98	1.80	1.67	0 56	1.58	2.35	2 46	33.52
1838..	3.07	2.77	3.09	2.62	3.32	2.55	1.20	4.26	9.87	5.02	3.95	0.80	42.52
1839..	0.98	3.11	1.18	7.73	4.27	2.25	3.32	5.70	2.00	2.50	1.71	6.35	41.10
1840..	3.12	2.57	4.55	4.60	2.23	2.78	2.93	4.00	2.12	4.48	11.63	4.15	49.16
1841..	6.00	1.60	3.50	8.82	1.90	1 95	2.10	4.20	2.86	3.80	4.55	5.77	47.05
1842..	0.80	3.20	3.35	3.50	2.90	5.30	1.82	4.44	3.25	0.80	4.45	5.30	39.11
1843..	2.20	6.08	6.17	3.88	1.60	4.61	2.15	6.88	0.98	4.82	3.40	3.92	46.69
1844..	3.68	2.42	6.00	0.20	2.72	1.40	2.17	2.62	3.53	5.80	3.15	3.85	37.54
1845..	4.58	4.25	3.83	1.23	2.82	2.05	3.28	1.82	2.23	4.00	10.25	5.98	46.32
1846..	3.12	2.95	2.73	1.23	2.02	2.25	2.51	1.80	1.30	1.35	4.17	4.52	29.95
1847..	3.28	4.70	4.77	2.20	2.03	4.09	2.65	6.45	6.64	1.05	5.12	3.95	46.93
1848..	2.30	3.90	4.05	1.40	6.30	1.73	1.35	3.10	3.55	5 10	2.25	5.95	40.98
1849..	0.35	1.15	7.35	0.90	3.10	1.45	0.85	6.25	1.25	8.10	5.50	4.05	40.30
1850..	4.59	2.52	5.32	4.82	6.63	2.77	2.70	5.30	7.15	2.10	3.32	6.76	53.98
1851..	1·30	4.20	3.88	9.37	3 31	1.80	3.09	1.27	3.50	4.43	5.51	2.65	44.31
1852..	4.85	2.85	4.45	10.18	1.95	2.35	3.28	7.63	1.65	2.19	3.47	3.09	47.94
1853..	2.44	5.30	2.27	3.78	5.63	0.30	3.64	9.40	3.80	3.92	4.43	3.95	48.86
1854..	2.91	4.87	2.84	6.63	4.33	2.47	3.70	0.58	3.86	2.08	6.80	4.64	45.71
1855..	7.22	4.67	1.18	4.28	1.20	3.09	4 15	1.46	1.13	4.61	5.27	5.93	44.19
1856..	5.32	0.80	1.33	4.37	7.10	2.90	4.02	11.11	4.90	2.70	3.33	4.28	52.16
1857..	5.36	2.45	3.09	10.83	5.57	2.02	5.53	7.18	2.56	4.50	2.52	5.26	56.87
1858..	3.28	2.30	2.18	5.18	3.89	8.09	4.56	7.03	5.02	3.03	3.38	4.73	52.67
1859..	5.93	4.05	7.64	3.36	3.63	7.89	1.58	4.72	4.40	3.28	3.75	6 47	56.70
1860..	1.89	3.85	4.70	1.97	2.35	8.01	5.90	4.3ᴼ	7.35	2.66	5.37	5.86	51.46
1861..	6.04	3.57	7.48	5.89	2.97	3.64	2.76	6.04	1.77	2.66	4.90	2.35	50.07
1862..	8.30	3.29	4.70	2.70	6.78	7.33	4.20	5.61	4.85		8.32	3.01	61.06
1863..	4.51	4.54	6.42	9.08	2 82	2.56	12.38	5.64	3.12	3.83	6.48	6.34	67.72
1864..	3.87	1.43	11.75	4.72	3.31	1.47	1.90	4.17	2.60	4.80	4.00	5.28	49.30
1865..	4.47	5.08	4.83	2.57	6.90	2.83	4.26	1.42	0.62	6.21	4.46	4.18	47.83
1866..	3.73	5.28	4.70	2.03	5.04	3.41	5.42	3.87	5.90	2.72	3.74	4.86	50.70
1867..	6.06	6.55	6.12	2 52	4.11	2.74	4.76	10.78	0.44	6.76	2.32	2.48	55.64
1868..	6.09	1.88	5.04	6.94	10.38	3.79	1.10	7.53	11.95	1.78	5.31	2.32	64.11
1869..	4.03	9.98	8.74	2.05	6.88	4.44	3.3ᴼ	2.19	5.18	6.71	3.74	9.04	66.28
1870..	8.16	7.03	4.88	8.42	2.58	7.59	4.01	1.57	0.67	6.80	4.40	3.62	59.73
1871..	2.77	3.72	4.68	4.23	5.69	5.67	2.87	3.31	1.37	5.51	5.38	3.13	48.33
1872..	2.43	2.68	3.98	3.24	3.95	4.81	4.48	10.48	7.37	4.98	4.64	5.00	58.04
1873..	6.69	3.74	4.54	3.81	4.92	0.65	3.25	6.46	2.78	5.43	7.34	5.33	54.94

TABLE No. 1—*Continued.*

Year.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	For the Year.
1874..	4.30	4.02	1.64	8.36	3.72	2.91	2.70	6.48	1.66	1.02	2.58	1.70	41.09
1875..	3.24	3.62	5.76	4.46	3.89	7.73	3.84	3.50	3.32	5.06	5.62	0.97	51.01
1876..	1.89	5.24	8.25	5.61	3.14	2.16	6.50	1.82	3.62	2.13	9.82	5.01	55.19
1877..	4.03	1.20	9.08	3.82	3.58	3.05	2.58	5.68	0.54	8.19	8.36	1.05	51.15
1878..	7.30	6.08	5.37	5.88	0.92	2.06	3.63	6.50	2.32	6.10	7.11	5.62	58.89
1879..	2.74	3.54	4.31	6.97	1.16	5.62	3.43	6.45	1.86	0.80	3.53	4.64	45.05
1880..	3.23	4.73	3.85	3.28	1.86	0.63	7.52	2.87	2.30	3.41	2.07	3.14	38.89
1881..	5.59	5·15	6.94	2.36	3.59	7.01	3.51	1.17	2.52	2.86	4.42	4.10	49.22
1882..	4.66	6.35	3.57	2.61	5.80	1.74	4.01	1.69	11.47	2.30	1.74	2.48	48.42
1883..	4.04	3.55	2.14	2.95	3.94	2.57	2.51	0.34	1.38	6.34	2.07	3.73	35.56
1884..	5.71	7.19	6.25	4.88	3.32	4.21	5.33	5.30	0.23	3.34	3.19	4.91	53.86
1885..	5.21	2.88	1.21	3.62	4.35	3.38	1.49	7.24	1.60	5.67	5.53	2.00	44.07
1886..	7.04	7.38	3.76	2.52	3.96	1.30	2.09	3.98	2.94	3.06	3.73	4.71	46.47
1887..	5.57	4.44	5.20	4.74	1.69	2.08	3.69	3.53	1.35	3.21	2.75	3.66	41.91
1888..	3.88	3.34	5.53	2.44	5.90	2.58	1.98	7.10	9.45	4.54	8.17	5.36	60.27
1889..	6.50	1.93	2.17	4.02	4.78	3.31	9.23	4.81	5.30	3.83	6.25	2.66	54.79
1890..	2.52	3.12	7.64	2.93	5.80	2.60	2.43	3.37	4.89	8.78	1.37	4.76	50.21
1891..	6.98	8.29	5.63	2.98	2.05	4.04	3.44	4.02	3.07	5.70	2.70	3.73	49.63
Totals	294.41	279.79	322.59	300.13	280.30	241.83	274.54	325.07	262.37	284.48	319.29	292.79	3477.66
Means	3.98	3.78	4.36	4.06	3.79	3.27	3.71	4.39	3.55	3.84	4.31	3.96	47.00

* Hall's gauge was 18 inches high, 12 inches diameter, and located 90 feet above tide-marsh. The snow was melted and measured.

† The Sewer and Water-Works gauges are 14.85 inches diameter, and the collections are weighed.

TABLE No. 2.

RAINFALL.—SUDBURY RIVER WATER-SHED—SIXTEEN YEARS.

NOTE.—Means of Observations at Several Places. Taken for 1875-86, from Special Report of May, 1887, on Capacity in Time of Drought, and for 1887-90, from Annual Reports of Water Board.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. TOTAL.	1875-90. MEAN.
Jan....	2.42	1.83	3.22	5.63	2.48	3.57	5.56	5.95	2.81	5.08	4.71	6.36	5.29	4.15	5.37	2.53	66.87	4.179
Feb....	3.15	4.21	0.74	5.97	3.56	3.98	4.65	4.55	3.86	6.55	3.87	6.28	4.78	3.68	1.65	3.51	64.99	4.062
Mar....	3.74	7.43	8.36	4.69	5.14	3.31	5.73	2.65	1.78	4.72	1.07	3.61	4.90	6.02	2.37	7.73	73.25	4.578
Apr....	8.23	4.20	3.44	5.79	4.72	3.10	2.00	1.42	1.84	4.40	3.60	2.23	4.26	2.43	3.41	2.65	53.12	3.320
May....	3.56	2.76	3.70	0.96	1.58	1.84	3.51	5.06	4.19	3.47	3.49	2.99	1.17	4.82	2.94	5.21	51.25	3.203
June...	6.24	2.04	2.42	3.88	3.79	2.14	5.39	1.66	2.40	3.44	2.87	1.47	2.65	2.54	2.80	2.03	47.76	2.985
July...	3.57	9.13	2.95	2.97	3.93	6.97	2.35	1.77	2.68	3.67	1.43	3.26	3.76	1.41	8.94	2.46	60.55	3.784
Aug....	5.53	1.72	3.68	6.94	6.51	4.01	1.36	1.67	0.74	4.65	7.18	4.10	5.28	6.22	4.18	3.86	67.63	4.227
Sept...	3.43	4.62	0.32	1.29	1.88	1.60	2.62	8.74	1.52	0.86	1.42	2.91	1.32	8.58	4.60	6.00	51.71	3.233
Oct....	4.85	2.24	8.52	6.42	0.81	3.74	2.95	2.07	5.60	2.48	5.10	3.23	2.83	4.99	4.99	10.51	70.60	4.413
Nov....	4.83	5.76	5.80	7.02	2.68	1.79	4.09	1.15	1.81	2.65	6.09	4.65	2.67	7.23	6.29	1.20	65.71	4.107
Dec....	0.94	3.62	0.87	6.37	4.34	2.83	3.96	2.30	3.55	5.17	2.72	4.97	3.88	5.39	3.14	5.31	59.36	3.710
Total...	45.49	49.56	44.02	57.93	41.42	38.18	44.17	39.39	32.78	47.14	43.55	46.06	42.70	57.46	49.95	53.00	732.80	45.800

January, 1875, to December, 1876, Lake Cochituate, with April to December, 1876, Westborough and Hopkinton, and June to December, 1876, Southborough and Marlborough. December, 1876, to January, 1883, Framingham, Southborough, Marlborough, Westborough and Hopkinton. January, 1883, to January, 1884, Framingham and Southborough. January, 1884, to January, 1890, Framingham and Westborough. January, 1890, to January, 1891, Framingham and Ashland.

TABLE No. 3.

YIELD OF THE SUDBURY RIVER WATER-SHED IN INCHES OF DEPTH—SIXTEEN YEARS—INCHES RAINFALL COLLECTED.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. TOTAL.	1875-90. MEAN.
Jan	0.18	1.15	1.17	3.23	1.25	2.00	0.74	2.21	0.60	1.77	2.20	2.61	4.62	1.88	4.96	2.24	32.81	2.05
Feb	2.41	2.28	1.53	3.97	2.76	2.98	2.49	3.87	1.66	4.74	2.18	7.73	4.56	3.25	1.93	2.46	50.82	3.18
Mar	2.86	7.91	8.59	6.26	4.16	2.45	7.14	5.06	2.87	6.75	2.80	3.67	5.12	5.78	2.39	6.50	80.31	5.02
Apr	5.26	5.68	4.13	2.81	5.38	2.02	2.67	1.50	2.33	4.92	3.13	3.36	4.52	4.57	2.43	3.24	57.96	3.62
May	2.12	2.03	2.48	2.49	1.99	0.92	1.72	2.39	1.67	1.84	2.38	1.28	1.80	2.91	1.57	2.44	31.94	2.00
June ...	1.50	0.38	1.03	0.87	0.71	0.30	2.31	0.91	0.52	0.72	0.73	0.35	0.71	0.73	1.13	0.98	13.90	0.87
July	0.57	0.33	0.36	0.23	0.28	0.31	0.49	0.15	0.21	0.40	0.11	0.21	0.20	0.21	1.13	0.19	5.39	0.34
Aug	0.71	0.72	0.22	0.85	0.70	0.21	0.26	0.10	0.14	0.46	0.43	0.17	0.38	0.68	2.55	0.23	8.82	0.55
Sept	0.36	0.32	0.10	0.28	0.24	0.14	0.34	0.53	0.16	0.08	0.21	0.20	0.19	1.99	1.42	0.79	7.35	0.46
Oct	1.15	0.42	1.13	0.92	0.13	0.18	0.33	0.53	0.33	0.15	0.60	0.26	0.34	3.57	2.19	4.05	16.28	1.02
Nov	2.25	1.88	2.45	2.92	0.35	0.35	0.68	0.36	0.35	0.30	2.03	1.16	0.64	4.76	3.35	2.10	25.94	1.62
Dec	1.04	0.81	2.30	5.67	0.82	0.31	1.38	0.56	0.34	1.65	2.09	1.82	1.15	5.43	4.00	1.78	31.16	1.95
Total..	20.42	23.91	25.49	30.49	18.77	12.18	20.56	18.10	11.19	23.78	18.92	22.82	24.23	35.75	29.06	26.99	362.66	22.67

TABLE No. 4.

PERCENTAGE OF RAINFALL COLLECTED—SUDBURY RIVER WATER-SHED—SIXTEEN YEARS.

NOTE.—For the years 1875-79, and 1887-90, figures are copied from Reports of Boston Water Board.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. MEAN.
Jan....	7.6	62.7	36.5	57.3	50.4	56.1	13.3	37.1	21.4	34.8	46.7	41.0	88.8	45.3	92.4	88.4	49.1
Feb....	76.5	54.2	200.9	66.5	77.4	74.9	53.6	85.1	42.9	72.4	56.4	123.1	95.3	88.3	116.4	70.3	78.2
Mar....	76.5	106.5	102.7	133.4	80.9	74.0	124.6	191.0	161.2	143.0	261.7	101.7	104.4	95.9	100.9	84.0	109.6
Apr....	162.9	145.4	120.3	48.5	114.1	65.1	133.5	82.2	126.3	111.7	86.8	151.0	106.0	188.8	71.4	122.3	109.1
May...	59.5	73.5	67.0	260.2	125.8	50.1	49.0	45.4	39.9	53.0	68.3	42.7	154.5	60.3	53.3	46.8	62.3
June..	24.0	18.8	42.5	22.5	18.8	14.0	42.8	54.7	21.7	20.9	25.5	23.9	26.9	28.7	40.3	48.3	29.1
July...	16.0	3.6	12.2	7.7	7.1	4.9	20.9	8.5	7.8	10.9	7.7	6.4	5.4	14.9	12.6	7.8	8.9
Aug...	12.8	42.0	5.9	12.2	10.8	5.2	19.1	6.0	19.0	9.9	6.0	4.1	7.2	10.9	61.2	6.1	13.0
Sept...	10.4	6.9	31.9	21.5	12.9	8.7	13.0	6.1	10.5	9.4	14.7	7.0	14.5	23.2	30.9	13.2	14.2
Oct....	23.8	18.6	13.2	14.3	15.6	4.8	11.1	25.6	5.9	6.0	11.8	8.0	12.0	71.4	61.6	38.6	23.1
Nov....	46.5	32.6	42.2	41.6	13.2	19.6	16.6	31.4	19.3	11.3	33.3	25.0	23.8	65.9	53.3	174.7	39.5
Dec.....	110.7	22.3	264.4	89.0	19.0	11.0	34.9	24.4	9.6	31.9	76.8	36.6	29.6	100.6	127.3	33.5	52.5
Mean..	44.9	44.2	57.9	52.6	45.3	31.9	46.5	45.9	34.1	50.4	43.4	49.5	56.7	62.2	58.2	50.9	49.5

Above are quotients obtained from tabulated yield, for each month, for each year, and for each sixteen months of the same name, by dividing by the tabulated rainfall for the corresponding time.

TABLE No. 5.

Evaporation from Water Surface in Inches—Sixteen Years.

	1876.	1877.	1878.	1879.	1880.	1875-1800 and 1881-84	1885.	1886.	1887.	1888.	1889.	1875-90.	
												Total.	Mean.
January........	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	*0.96	15.36	0.96
February.......	*1.05	*1.05	*1.05	*1.05	*1.05	*1.05	*1.05	*1.05	*1.05	*1.45	*1.05	16.80	1.05
March.........	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	*1.70	27.20	1.70
April.........	*2.98	*2.98	*2.98	*2.98	*2.98	*2.98	*2.94	3.12	3.07	2.78	2.84	47.57	2.97
May...........	*4.45	4.05	4.14	5.89	5.22	*4.45	3.77	4.45	4.48	3.35	4.57	71.42	4.46
June..........	5.44	5.08	5.26	5.32	5.46	*5.55	7.01	5.25	5.05	5.98	3.94	88.69	5.64
July..........	7.60	4.82	6.04	6.41	6.82	*5.98	7.09	6.39	5.96	5.57	5.04	95.72	5.98
August........	6.21	4.40	4.33	5.23	5.34	*5.50	7.41	5.80	6.20	5.81	4.25	87.98	5.50
September.....	3.48	4.08	4.04	3.80	4.04	*4.20	5.13	4.55	4.57	3.91	3.08	65.88	4.12
October.......	3.12	2.51	3.82	2.99	2.79	*3.11	2.70	4.13	3.61	3.27	3.13	50.52	3. 6
November......	0.66	*2.23	*2.23	*2.23	2.60	*2.23	*2.23	2.69	3.00	2.71	1.95	35.94	2.25
December......	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	*1.51	24.16	1.51
Total........	39.06	35.97	37.76	40.07	40.47	39.22	43.63	40.80	41.51	38.60	34.05	627.24	39.20

* From curve of mean evaporation, so adjusted as to vary very slightly from the means of observations. Unstarred numbers are from observations of Chestnut Hill Reservoir.

TABLE No. 6.

YIELD OF THE SUDBURY RIVER WATER-SHED IN MILLIONS OF GALLONS—SIXTEEN YEARS.

	*1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. Total.	Mean.
Jan..	248.0	1 550.3	1 586.9	4 363.1	1 698.2	2 716.0	964.9	2 892.2	780.5	2 319.9	2 878.5	3 405.3	6 036.0	2 454.3	6 486.0	2 924.0	43 305.70	2 706.61
Feb..	3 257.×	3 004.5	2 066.9	5 367.9	3 747.9	4 054.4	3 235.5	5 060.3	2 174.4	6 197.3	3 830.9	10 104.3	5 966.3	4 253.3	2 517.4	3 219.6	67 172.60	4 198.29
Mar..	3 867.2	10 691.1	11 604.1	8 454.4	5 631.4	3 332.2	9 333.6	6 617.5	3 755.0	8 824.3	3 645.6	4 798.8	6 685.4	7 547.5	3 120.2	8 492.0	106 440.70	6 652.54
Apr..	7 113.2	7 680.4	6 554.1	3 793.2	7 313.7	2 743.0	3 487.7	1 956.8	3 044.5	6 437.3	4 094.8	4 392.5	5 909.9	5 907.5	3 180.8	4 229.1	76 928.50	4 808.03
May..	2 863.1	2 744.0	3 354.2	3 361.6	2 701.4	1 247.0	2 249.6	3 010.9	2 185.3	2 402.1	3 114.4	1 679.1	2 331.6	3 805.6	3 050.3	3 185.4	42 305.50	2 644.09
Jun..	2 029.1	517.5	1 393.0	1 179.8	970.2	411.7	3 017.8	1 193.3	676.9	939.0	960.7	456.2	932.5	950.8	1 473.5	1 280.8	18 381.80	1 149.05
Jul..	774.7	441.1	486.1	309.5	381.7	427.7	641.3	201.1	268.9	521.5	144.7	269.7	267.6	273.6	477.0	250.0	7 139.16	446.20
Aug..	954.0	976.9	291.9	1 146.3	958.3	288.2	345.2	128.9	183.1	598.5	560.2	219.4	499.4	884.6	338.1	306.9	11 679.89	729.99
Sep..	483.8	430.2	139.0	375.1	330.3	188.4	444.9	691.5	205.9	99.1	273.1	264.9	250.4	674.8	1 957.7	1 031.6	9 670.70	604.42
Oct..	1 537.5	563.0	1 522.4	1 244.3	171.5	246.5	432.6	697.7	433.2	193.8	742.5	339.7	443.2	4 659.8	2 867.5	5 296.5	21 451.70	1 340.73
Nov..	3 038.0	2 537.8	3 307.3	3 949.0	482.5	481.7	891.0	472.3	461.7	395.3	2 656.3	1 518.5	831.5	6 222.1	4 378.8	2 740.1	34 363.90	2 147.74
Dec..	1 407.3	1 093.1	3 108.9	7 658.8	1 121.8	424.8	1 806.9	733.6	451.1	156.0	2 736.7	2 377.3	1 499.7	7 092.9	1 223.7	2 321.5	41 214.10	2 575 88
Total	27 593.7	32 309.9	34 444.8	41 202.0	25 528.9	16 561.6	26 876.0	23 656.5	14 620.5	31 084.1	24 718.4	29 831.7	31 663.5	46 717.3	37 971.0	35 277.4	450 057.25	30 003.58

*Area of water-shed: 1875-78, 77,764 square miles; 1879-80, 78,238 square miles; 1881-90, 75,199 square miles.

TABLE No. 7.

YIELD OF THE SUDBURY RIVER WATER-SHED IN CUBIC FEET PER SECOND—SIXTEEN YEARS.

	*1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1895.	1886.	1887.	1888.	1889.	1890.	1875-90. SUM.	1875-90. MEAN.
Jan	12.38	77.38	79.20	217.72	84.76	135.56	48.26	144.33	38.96	115.79	143.67	169.96	301.26	122.53	323.72	115.94	2161.41	135.09
Feb	180.02	164.57	114.21	206.62	207.10	216.31	179.89	279.62	120.15	380.64	157.54	558.57	329.13	296.93	139.11	177.90	......	229.94
Mar	193.01	533.60	579.17	421.96	282.06	166.31	465.85	330.30	187.41	440.43	182.93	239.51	333.67	376.70	155.73	423.84	5312.52	332.03
April	365.86	396.11	288.00	193.63	377.20	141.47	179.88	100.92	157.02	382.00	211.19	226.54	304.40	307.77	164.05	218.11	3967.53	247.97
May	142.90	136.95	167.41	167.78	134.83	62.94	112.28	154.28	109.07	119.80	135.44	83.80	117.37	159.93	102.33	158.99	2111.49	131.97
June	104.65	226.69	71.84	60.85	50.04	21.23	155.64	61.54	34.91	48.43	49.55	23.63	46.09	49.04	75.99	66.06	948.18	59.26
July	38.67	22.02	24.26	15.45	19.05	21.35	32.16	10.04	13.42	26.03	7.22	23.46	13.36	13.66	73.72	12.48	356.32	22.27
Aug	47.61	48.76	14.57	57.21	47.83	14.38	17.23	6.43	9.14	29.87	27.96	10.95	24.93	44.15	166.61	15.32	582.95	36.43
Sept	24.95	22.19	7.17	19.35	17.03	9.72	22.95	35.66	10.62	5.11	14.08	13.66	12.91	134.84	95.81	53.20	498.76	31.17
Oct	77.74	28.10	75.98	62.10	8.66	12.30	21.59	34.82	21.62	9.67	39.06	16.95	22.12	232.57	143.12	264.35	1070.67	66.92
Nov	156.68	130.89	170.57	203.67	24.88	24.84	45.93	24.36	23.81	20.39	137.00	78.32	42.88	320.90	225.88	141.32	1772.29	110.77
Dec	70.24	64.56	155.17	382.26	65.99	21.20	90.18	36.61	22.51	107.61	136.59	118.65	74.85	354.01	260.72	115.87	2057.02	128.56
Mean	116.97	136.59	146.01	174.65	108.22	70.01	113.93	100.28	61.98	131.41	104.78	126.46	134.22	197.49	160.96	149.54	......	127.10

* Area of Water-Shed: 1875-78, 77.764 square miles. 1879-80, 78.238 square miles. 1881-90, 75.199 square miles.

TABLE No. 8.

YIELD OF THE SUDBURY RIVER WATER-SHED IN MILLIONS OF GALLONS PER SQUARE MILE—SIXTEEN YEARS.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. TOTAL.	1875-90. MEAN.
January	3.199	19.936	20.407	56.098	21.705	34.714	12.857	38.461	10.379	30.850	38.278	45.284	80.267	32.645	86.251	38.883	570.204	35.638
February	41.894	39.665	26.580	69.028	47.904	51.823	43.294	67.292	28.915	82.412	37.911	131.421	79.210	66.562	33.477	42.813	883.201	55.200
March	49.729	137.481	149.222	104.719	72.237	42.592	124.119	88.003	49.983	117.346	48.745	63.815	88.903	100.366	41.493	112.928	1 395.633	87.227
April	91.472	98.766	71.809	48.779	93.480	35.059	46.380	26.022	40.487	85.604	54.453	58.412	78.590	79.365	42.296	56.239	1 007.205	62.950
May	36.819	35.286	43.134	43.228	31.529	15.938	20.916	40.039	29.061	31.943	41.415	22.329	31.271	60.604	27.265	43.360	555.137	34.696
June	26.093	6.655	17.914	15.173	12.401	5.262	40.130	15.869	9.001	12.487	12.775	6.093	12.401	12.644	19.595	17.032	241.526	15.095
July	9.962	5.672	6.251	3.979	4.878	5.467	8.668	2.674	3.576	6.935	1.924	3.586	3.558	3.636	19.641	3.325	93.634	5.852
August	12.268	12.562	3.754	14.741	12.247	3.684	4.600	1.714	2.435	7.959	7.450	2.917	6.641	11.763	44.330	4.081	153.196	9.575
September	6.221	5.532	1.787	4.823	4.221	2.408	5.916	9.195	2.738	1.318	3.632	3.523	3.329	34.640	24.704	13.718	127.705	7.982
October	20.029	7.240	19.672	16.092	2.191	3.150	5.752	9.278	5.761	2.577	10.406	4.517	5.893	61.968	38.132	70.433	262.901	17.681
November	39.067	32.635	42.530	50.782	6.166	6.156	11.848	6.281	6.199	5.256	35.324	20.193	11.057	83.743	58.229	36.438	450.844	28.178
December	18.097	14.057	39.979	98.481	14.339	5.429	24.028	9.755	5.999	28.071	36.393	31.613	19.943	94.321	69.465	30.871	541.441	33.840
Total	354.840	415.487	442.039	529.833	326.298	211.682	357.398	314.585	194.424	413.386	328.706	396.706	421.063	621.249	504.940	469.121	6 302.626	393.914

TABLE No. 9.

YIELD OF THE SUDBURY RIVER WATER-SHED IN CUBIC FEET PER SECOND PER SQUARE MILE—SIXTEEN YEARS.

| | 1875. | 1876. | 1877. | 1878. | 1879. | 1880. | 1881. | 1882. | 1883. | 1884. | 1885. | 1886. | 1887. | 1888. | 1889. | 1890. | 1875–90. | |
																	SUM.	MEAN.
Jan	0.159	0.995	1.019	2.800	1.043	1.743	0.642	1.920	0.518	1.540	1.910	2.260	4.006	1.629	4.305	1.941	28.459	1.779
Feb	2.315	2.116	1.469	3.814	2.647	2.765	2.392	3.718	1.598	4.397	2.045	7.428	4.377	3.011	1.850	2.366	.....	3.023
Mar	2.482	6.862	7.448	5.426	3.605	2.126	6.195	4.392	2.492	5.857	2.433	3.185	4.437	5.009	2.071	5.636	69.657	4.354
Apr	4.718	5.094	3.703	2.516	4.821	1.804	2.392	1.312	2.088	4.415	2.808	3.013	4.053	4.093	2.182	2.900	51.946	3.247
May	1.838	1.761	2.153	2.158	1.723	0.796	1.493	1.998	1.450	1.594	2.067	1.114	1.561	2.026	1.361	2.114	27.707	1.732
June....	1.346	0.343	0.924	0.783	0.640	0.271	2.070	0.818	0.464	0.644	0.659	0.314	0.640	0.652	1.011	0.878	12.436	0.779
July....	0.497	0.283	0.312	0.199	0.243	0.273	0.428	0.133	0.178	0.346	0.096	0.179	0.178	0.182	0.980	0.166	4.673	0.292
Aug....	0.612	0.627	0.187	0.736	0.611	0.184	0.229	0.086	0.122	0.397	0.372	0.146	0.331	0.587	2.216	0.204	7.646	0.478
Sept....	0.321	0.285	0.092	0.249	0.218	0.124	0.305	0.474	0.142	0.068	0.187	0.182	0.172	1.746	1.274	0.708	6.586	0.412
Oct....	1.000	0.361	0.977	0.799	0.109	0.157	0.287	0.463	0.288	0.129	0.519	0.225	0.294	3.093	1.903	3.515	14.120	0.882
Nov....	2.015	1.683	2.193	2.619	0.318	0.314	0.611	0.324	0.317	0.271	1.822	1.041	0.570	4.267	3.003	1.879	23.252	1.453
Dec....	0.903	0.702	1.995	4.916	0.716	0.271	1.199	0.487	0.299	1.431	1.816	1.578	0.995	4.708	3.467	1.541	27.024	1.680
Mean...	1.504	1.756	1.878	2.246	1.383	0.895	1.515	1.334	0.824	1.747	1.393	1.682	1.785	2.626	2.140	1.989	.....	1.669

TABLE No. 10.

CALCULATED YIELD OF THE SUDBURY RIVER WATER-SHED ON THE BASIS OF THE LAND AND WATER SURFACE OF 1890-91, IN MILLIONS OF GALLONS PER SQUARE MILE—SIXTEEN YEARS.

NOTE.—Water Surface in 1890-91 was about 3¼ per cent., varying slightly in different months, land nearly 97 per cent.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90.	
																	TOTAL.	MEAN.
Jan.	3.538	19.862	20.693	56.477	21.751	34.760	13.304	38.651	10.605	31.128	38.374	45.524	80.261	32.658	86.249	38.883	572.608	35.798
Feb.	41.812	39.895	26.096	69.277	47.877	51.818	43.395	67.266	29.013	82.455	37.940	134.291	79.181	56.557	33.473	42.813	883.161	55.198
Mar.	49.515	136.912	148.718	107.868	72.159	42.534	123.949	87.766	49.795	117.154	48.637	63.691	88.898	100.361	41.491	112.928	1 392.376	87.023
Apr.	90.144	97.582	70.834	48.779	93.049	34.930	46.166	25.874	40.296	85.408	54.330	57.957	78.579	79.338	42.291	56.239	1 001.816	62.613
May.	36.016	34.496	42.379	41.705	33.412	15.638	29.786	39.961	28.959	31.776	41.321	22.013	31.237	50.597	27.250	42.358	548.704	34.294
June	25.875	16.760	14.562	14.562	12.337	4.900	40.011	15.627	8.806	12.327	12.567	5.600	12.393	12.617	19.684	17.032	236.304	14.769
July	9.172	5.637	5.660	3.098	4.389	5.478	8.319	2.419	3.328	6.792	1.623	3.271	3.552	3.626	8.642	4.081	89.713	5.607
Aug.	12.095	6.019	3.098	16.194	3.966	3.556	5.478	1.349	1.893	7.898	7.418	2.681	6.649	11.764	44.415	3.325	150.349	9.397
Sept.	5.944	11.231	0.840	4.071	4.071	2.175	4.273	9.600	2.367	1.168	3.464	3.295	3.381	34.597	24.702	13.718	124.756	7.797
Oct.	20.177	5.732	20.814	16.502	1.595	3.242	5.746	9.147	6.093	2.505	10.490	4.369	5.893	61.975	38.139	70.433	284.297	17.769
Nov.	39.159	6.911	42.925	51.278	6.181	6.004	11.981	6.143	6.005	5.271	35.402	20.257	11.041	82.741	68.234	36.438	452.440	28.277
Dec.	17.673	14.399	39.205	98.334	14.567	5.563	24.135	9.778	6.356	28.810	36.363	31.657	19.961	94.314	69.463	30.871	541.439	33.840
Total ...	351.110	411.966	438.389	527.161	323.689	210.609	356.769	313.583	193.416	412.692	327.937	394.606	421.026	621.145	504.033	460.119	6 277.963	392.373

TABLE No. 11.

CALCULATED YIELD OF ONE SQUARE MILE OF LAND SURFACE IN MILLIONS OF GALLONS—SIXTEEN YEARS.

NOTE.—Deduced from observed yield of the Sudbury River Water-Shed by making allowance for the yield of the portions covered by water.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875–90. Total.	Mean.
January	2.762	20.029	20.045	55.611	21.589	34.391	10.970	36.966	9.746	29.707	37.435	43.827	80.491	31.860	86.686	39.290	561.305	35.082
February	41.997	39.371	27.195	68.709	48.024	51.849	42.729	67.495	28.319	82.000	37.557	135.806	79.681	56.932	34.275	42.818	884.757	53.297
March	50.005	138.211	149.808	109.814	72.589	43.038	125.826	90.247	51.479	119.405	50.711	64.753	90.086	101.241	42.533	113.208	1 412.984	88.311
April	93.149	100.260	73.039	48.777	95.252	36.078	48.379	27.494	42.898	87.533	53.874	60.532	80.604	92.384	43.425	58.413	1 033.581	64.596
May	57.824	36.530	44.080	45.113	37.217	18.252	31.403	40.988	30.138	33.490	42.946	23.675	34.571	51.474	29.199	43.382	580.284	36.268
June	26.365	7.921	19.347	15.919	13.369	7.724	41.518	18.561	11.049	14.062	15.152	8.118	14.302	15.175	20.974	19.792	269.738	16.859
July	10.960	5.235	5.996	5.075	6.051	5.394	10.930	5.068	5.453	8.439	5.128	4.801	7.015	6.288	17.959	5.584	114.296	7.142
August	12.494	14.307	6.906	14.157	11.996	4.476	6.900	3.690	4.512	8.679	7.809	3.791	5.429	11.921	45.991	5.201	167.319	10.482
September	6.899	5.269	3.070	5.826	3.232	3.682	6.872	7.292	4.025	3.176	5.732	4.375	5.408	33.017	24.641	13.123	137.319	8.582
October	19.832	17.673	17.945	15.346	3.262	2.786	5.986	10.080	4.816	2.968	9.483	5.054	6.538	63.077	38.777	68.435	282.078	17.630
November	38.949	31.555	42.153	50.161	6.123	6.707	11.271	7.013	6.469	5.203	34.295	19.779	11.625	82.890	57.660	98.332	460.175	28.136
December	18.636	13.621	40.963	98.820	13.356	4.958	23.492	9.642	5.310	27.595	36.888	30.609	19.220	96.250	70.899	29.644	538.983	33.686
Total	369.572	419.972	448.776	533.328	334.060	219.335	366.168	324.486	204.044	422.247	339.430	405.170	434.960	631.459	512.909	477.222	6 433.138	402.071

TABLE No. 12.

CALCULATED YIELD OF ONE SQUARE MILE OF LAND SURFACE IN CUBIC FEET PER SECOND—SIXTEEN YEARS.

NOTE.—Deduced from observed yield of the Sudbury River Water-Shed by making allowance for the yield of the portions covered by water.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90. SUM.	1875-90. MEAN.
Jan....	0.138	1.000	1.000	2.776	1.078	1.716	0.548	1.845	0.486	1.483	1.868	2.187	4.017	1.590	4.322	1.961	28.015	1.751
Feb....	2.321	2.101	1.503	3.797	2.654	2.766	2.361	3.780	1.565	4.375	2.075	7.594	4.403	3.037	1.894	2.366	......	3.029
Mar....	2.496	6.898	7.480	5.481	3.623	2.148	6.280	4.504	2.569	5.960	2.531	3.332	4.495	5.053	2.123	5.650	70.523	4.408
Apr....	4.804	5.171	3.767	2.516	4.913	1.861	2.495	1.417	2.187	4.514	2.842	3.122	4.157	4.246	2.240	3.013	53.304	3.331
May...	1.888	1.823	2.200	2.252	1.858	1.911	1.567	2.046	1.504	1.672	2.143	1.182	1.725	2.569	1.052	2.165	28.962	1.810
June...	1.360	0.408	0.998	0.821	0.689	0.398	2.141	0.957	0.570	0.725	0.862	0.419	0.788	0.783	1.082	1.021	13.912	0.869
July...	0.547	0.261	0.349	0.253	0.302	0.269	0.540	0.953	0.272	0.421	0.266	0.240	0.250	0.314	0.696	0.279	5.703	0.356
Aug....	0.624	0.714	0.203	0.707	0.590	0.223	0.344	0.184	0.240	0.433	0.399	0.189	0.371	0.505	2.295	0.260	8.371	0.523
Sept...	0.340	0.271	0.159	0.300	0.270	0.190	0.354	0.374	0.208	0.164	0.297	0.226	0.279	1.703	1.271	0.677	7.082	0.443
Oct....	0.990	0.383	0.496	0.766	0.163	0.139	0.299	0.503	0.240	0.148	0.473	0.252	0.327	3.148	1.935	3.416	14.079	0.880
Nov....	2.009	1.627	2.174	2.587	0.316	0.346	0.591	0.362	0.334	0.268	1.760	1.020	0.600	4.275	2.973	1.977	23.217	1.451
Dec....	0.930	0.680	2.044	4.932	0.667	0.247	1.172	0.481	0.267	1.377	1.841	1.530	0.909	4.734	3.539	1.480	26.901	1.681
Mean...	1.524	1.775	1.902	2.261	1.416	0.927	1.552	1.375	0.865	1.785	1.439	1.718	1.844	2.660	2.174	2.023	......	1.703

TABLE No. 13.

CALCULATED YIELD OF ONE SQUARE MILE OF WATER SURFACE IN MILLIONS OF GALLONS.—SIXTEEN YEARS.

NOTE.—From comparison of observed monthly rainfall with observed or assumed monthly evaporation from a water surface.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875–90. Total.	1875–90. Mean.
Jan.....	25.373	15.120	39.206	81.194	26.381	45.289	79.907	86.737	32.151	71.687	65.170	93.932	73.686	55.438	76.640	27.285	895.196	55.950
Feb.....	36.495	54.917	−5.405	85.556	43.665	50.920	62.494	60.756	48.921	95.496	48.921	90.891	64.823	45.793	10.514	42.665	837.412	52.338
Mar.....	35.438	99.680	115.690	61.945	59.783	28.067	70.036	16.492	1.390	52.484	−10.949	33.193	55.612	75.076	11.557	104.881	890.290	50.018
Apr.....	4.345	21.150	7.907	48.831	30.169	2.172	−17.031	−20.090	−19.725	24.765	10.862	−15.554	20.768	−6.169	9.906	−5.822	96.487	6.030
May.....	−15.467	−29.318	−6.048	−55.334	−74.920	−58.810	−16.319	10.705	−4.603	−17.031	−4.953	−25.286	−63.693	25.634	−28.240	13.208	−350.477	−21.905
June...	11.991	−59.088	−56.568	−23.913	−26.607	−75.111	−2.694	−67.534	−54.743	−36.582	−72.035	−65.779	−41.709	−59.870	−19.812	−61.173	−711.227	−44.452
July...	−41.888	28.397	−32.481	−53.335	−43.047	7.873	−63.085	−73.182	−57.350	−40.232	−98.451	−40.406	−38.233	−72.382	67.690	−61.173	−611.280	−38.205
Aug.....	0.521	−78.031	−12.478	45.306	22.228	−23.148	−71.983	−66.613	−82.810	−14.772	−3.910	−29.544	−15.988	7.212	−1.303	−28.414	−363.727	−22.108
Sept...	−13.382	19.707	−65.292	−47.774	−33.402	−42.352	−27.511	78.917	−46.576	−58.132	−64.388	−28.688	−66.481	81.246	26.503	31.282	−246.222	−15.389
Oct.....	30.230	−15.276	104.359	50.346	−37.903	16.510	−2.694	−18.004	43.273	−10.949	40.058	−15.554	−13.469	29.891	19.551	128.603	348.981	21.811
Nov.....	45.185	88.701	62.094	83.314	7.855	−14.164	32.342	−18.821	−7.299	7.212	67.169	33.975	−5.735	78.465	74.989	−17.900	517.382	32.336
Dec.....	−0.906	30.669	−11.122	84.409	49.251	22.905	42.543	13.660	35.453	63.606	21.028	60.217	41.188	67.516	28.327	66.039	611.783	38.236
Total...	108.964	182.528	139.862	350.548	23.449	−39.849	86.005	3.028	−111.919	137.552	−1.478	91.497	20.769	327.850	276.322	239.481	1 834.598	114.662

TABLE No. 14.

CALCULATED YIELD ON ONE SQUARE MILE OF WATER SURFACE IN CUBIC FEET PER SECOND—SIXTEEN YEARS.

NOTE.—From comparison of observed monthly rainfall with observed or assumed monthly evaporation from a water surface.

	1875.	1876.	1877.	1878.	1879.	1880.	1881.	1882.	1883.	1884.	1885.	1886.	1887.	1888.	1889.	1890.	1875-90 SUM.	1875-90 MEAN.
Jan.	1.266	0.755	1.957	4.052	1.317	2.260	3.998	4.329	1.605	3.578	8.253	4.698	3.678	2.767	3.925	1.362	44.680	2.792
Feb.	2.017	2.930	0.299	4.728	2.412	2.717	3.453	3.357	2.703	5.095	2.703	5.022	3.842	2.443	0.881	2.358	......	2.867
Mar.	1.769	4.970	5.774	2.593	2.984	1.401	3.496	0.823	2.069	2.619	−0.546	1.657	2.776	3.747	0.577	5.235	39.943	2.496
Apr.	0.224	1.091	0.408	2.519	1.556	0.112	−0.878	−1.036	−1.017	1.277	−0.560	−0.802	1.071	−0.319	0.611	−0.360	4.976	0.311
May	−0.772	−1.463	−0.302	−2.762	−3.739	−2.935	−0.139	−0.534	−0.230	−0.850	−0.247	−1.202	−3.179	−1.279	−1.409	0.695	−17.493	−1.093
June	0.618	−3.047	−2.917	−1.233	−1.372	−3.874	−3.149	−3.483	−2.823	−1.887	−3.715	−3.392	−2.161	−3.688	−1.022	−3.155	−36.681	−2.293
July	−2.090	1.417	−1.621	−2.662	−2.149	−3.303	−3.593	−3.653	−2.862	−2.008	−4.914	−2.017	−1.908	−3.613	3.378	−1.418	−30.509	−1.907
Aug.	0.026	−3.895	−0.623	−2.961	−1.109	−1.155	−1.419	−3.325	−4.133	−0.737	−0.195	−1.475	−0.798	−0.065	−0.065	−1.613	−17.635	−1.103
Sept.	−0.690	1.016	−3.367	−2.464	−1.723	−1.184	−0.134	−4.070	−2.492	−2.908	−3.321	−1.474	−2.913	4.190	1.367	6.419	−12.609	−0.794
Oct.	1.509	−0.762	5.209	−2.513	−1.892	−0.824	1.613	−0.899	−2.160	−2.160	1.999	−0.776	−0.672	1.492	0.976	−0.923	17.418	1.089
Nov.	2.330	4.575	3.202	4.297	0.405	−0.730	1.668	−0.971	−0.376	0.372	3.464	1.752	−0.296	4.047	3.868	3.296	26.684	1.668
Dec	−0.494	1.880	−0.555	4.213	2.458	1.143	2.123	0.682	1.769	3.175	1.050	3.005	2.056	3.370	1.414	3.296	30.534	1.908
Mean..	0.462	0.772	0.593	1.486	0.009	−0.168	0.365	0.013	0.474	0.581	−0.006	0.388	0.088	1.386	1.171	1.015	......	0.486

TABLE No. 15.

AVERAGE YIELD OF THREE WATER-SHEDS IN MILLIONS OF GALLONS PER SQUARE MILE.

MONTH.	LAKE COCHITUATE.			SUDBURY RIVER.		MYSTIC LAKE.	COCHITUATE AND SUDBURY.	COCHITUATE, SUDBURY AND MYSTIC.
	1863-1890, Inclusive.	1875-1890, Inclusive.	1878-1890, Inclusive.	1875-1890, Inclusive.	1878-1890, Inclusive.	1878-1890, Inclusive.	1875 to 1890, Inclusive.	1878 to 1890, Inclusive.
January	33.693	32.512	36.783	35.638	40.513	34.864	34.075	37.387
February	45.688	49.128	52.336	55.200	59.621	53.213	52.164	55.056
March	63.449	69.877	66.396	87.227	81.477	65.804	78.552	71.226
April	52.681	47.760	44.619	62.950	57.320	46.382	53.356	49.107
May	32.967	25.294	24.641	34.696	33.839	32.737	29.995	30.406
June	14.482	12.733	11.789	15.095	14.681	18.455	13.913	14.975
July	10.093	7.345	6.706	6.852	5.619	10.249	6.599	7.491
August	13.238	11.998	12.672	9.575	9.886	11.940	10.786	11.399
September	13.406	12.141	12.833	7.963	8.782	9.110	10.061	10.242
October	18.732	17.022	17.370	17.681	18.108	14.863	17.352	16.797
November	26.906	27.156	24.714	28.178	25.893	23.755	27.667	24.787
December	29.988	33.067	35.124	33.840	36.100	31.160	33.453	34.128
Total	355.323	346.033	345.983	393.914	391.489	351.832	369.973	363.001

DISCUSSION.

JAMES B. FRANCIS, Past President Am. Soc. C. E.—The recent great rains and floods in the West and the comparative drought in the East have led me to consider if this condition can be explained on any general principles. I suppose the rainfall depends on the evaporation caused by the heat of the sun, which I take to be uniform, and that the evaporation and rainfall, taking the earth as a whole, are constant quantities, although we find practically as to a particular locality very great inequality. We have had in the West enormous rains and floods; in the East at the same time the rainfall was extremely small. I account for it thus: The water evaporated must fall somewhere, but is irregularly distributed, depending on the currents of air; if there is an excess in some places there must be a corresponding deficiency in some other places.

A. FTELEY, M. Am. Soc. C. E.—The results shown in the paper just read have evidently been collected with great care, but it omits one point which is of importance in the computation of the amount of storage room necessary in connection with a given water-shed and a given daily consumption. I refer to the fact that the total amount drawn from reservoirs in a year of drought, when they are partly or wholly emptied, is greater than the apparent capacity of the basins. I have observed several cases where the excess was from 20 to 30 per cent. and more. In New York, last year, the storage reservoirs were entirely exhausted and the amount of water drawn from them, exclusive of the natural flow of the streams, exceeded by more than 30 per cent. the visible capacity.

This result is obviously due to the fact that the ground around the reservoirs fills up at the same time as their level rises, and that the amount of water thus stored in the ground gradually returns to the reservoir as its surface is lowered. The amount of water stored by these means necessarily varies with the nature of the ground, being practically *nil* for rocky surfaces, and increasing with the porosity of the materials of which it is formed.

CLEMENS HERSCHEL, M. Am. Soc. C. E.—The point just made by Mr. Fteley is one of the two that I wished to make about the paper under discussion. The Boston observations were, I believe, the first made on the yield of drainage areas, and, being the first, they have been worked up with more care and more accuracy than any others we have. I think, also, that we have had, until recently, only the observations made on the Boston and the New York water supply drainage areas to go from in this country. Now, in the course of time, the computations and tables that have been made, have gone into more and more refinements, which is, of course, conducive to an advance of

knowledge. But the moment we pretend to work from observations of this kind and achieve precise results, the more essential it is that absolutely no modifying causes be left out of the computation. It thus has come about that this matter of the yield of a reservoir, being more than its capacity when measured from a topographical map, has become one of material weight.

The two papers we have recently had on this subject, that of Mr. Stearns and of Mr. FitzGerald, are notable papers, but embodied in them is the underlying defect which Mr. Fteley and I have pointed out.

To illustrate, I will take the paper of Mr. Stearns, who computes that to yield from a drainage area the moderate quantity of 800 000 gallons per square mile, it is necessary to have something like two hundred and twenty-five days' visible supply in store. A storage capacity of that magnitude is, however, rarely found on any city water-works in the United States. A result of that sort immediately put me on my guard against the whole series of results found, and, examining into the matter, I judged that the reason such results were arrived at was simply that the point had not been considered, that the water as it rises, elevates with it the whole water table of the country surrounding the reservoir. So that the reservoir volume, as computed from a topographical map, does not represent the true storage volume; the true storage volume is this computed volume, together with what may be called the invisible storage, which latter is under ground. Where, in the one case, we might compute that we had got only one hundred and twenty days' supply, we would have in reality one hundred and sixty to one hundred and seventy days' supply in store.

The second point I alluded to is in the way of questioning whether to accept the results founded solely on the drainage areas of Boston and New York, as giving results applicable in any and all cases. They are composed in the main of hills, not of a mountainous character; lie near the sea-shore, with ample expanse of meadow and farm in the lower portions. Being accidentally of like character, the impression has grown up and has been allowed to prevail, that because these two give concordant results, no others need be expected. They give a yield of something like 50 or 51 or 49 per cent. of the total rainfall, varying very largely during the different years, and being even unlike during years of the same rainfall; for, as we know, the distribution of rainfall during the year is of the utmost importance. To compute the yield of any stream from such records, and from the rainfall of the section of country in which that stream is situated, thus becomes an uncertain matter. The thing we are all after is knowledge of the flow of a given stream in cubic feet per second; of what our western friends call "the run-off," and I imagine, that if the measurement of cubic feet per

second had been as simple a matter as measuring rainfall, we should now
have a much greater knowledge on the subject than we actually have.

I hope, as time goes on, cubic feet per second will be measured
more and more, and it will follow that inches of rainfall will lose the
importance that is now given them.

To illustrate in figures what I have just stated, I will read some
statistics which I have gathered :

[From the " Annales des Ponts et Chaussées."]

JANUARY, 1892.

Per Cent.

Durance, above Mirabeau, seven years' record........ 74
"   "   Avignon,   "   "   67
Three tributaries of La Loire........................ 71
Granit de Morvan.................................... 71
Reservoir Gondrexange 53
Portion of La Seine.......... 53
Reservoir Freyberg 45

While, as is well known, the general result on the Boston and New
York drainage areas is not quite 50 per cent.

June 1st, 1891, to June 1st, 1892, was a very dry year in the vicinity
of New York. The Croton water-shed yielded in that twelve months
15.74 inches of water; the yield of the Sudbury was, during the same
time, 15.63 inches; the minimum on record on the Croton, 1880, being
15.33 inches.

During the same twelve months, the Pequannock water-shed, above
the intake dam, near Charlottsburgh, situated in the same range of
rainfall•and of drought, but consisting largely of mountain slopes,
yielded 23.23 inches of water, being nearly 50 per cent. more than the
rate of yield from the Croton or the Sudbury water-shed.

I will say in this connection that it would be more scientific to
divide rainfall and yields of streams into periods from June 1st to
June 1st, than to divide the year in the usual way, from January 1st to
January 1st, to get true annual yields.

FREDERIC P. STEARNS, M. Am. Soc. C. E.—As a part of my regular
work, I have occasion very frequently to estimate the capacity of pro-
posed sources of water supply. In making such estimates, I have
always found the accurate records of the flow of streams, rainfall and
evaporation, made in connection with the Boston Water Works, and for
the most part under the supervision of Mr. FitzGerald, of the greatest
value. I wish to emphasize one point that has already been brought
out by Mr. FitzGerald, viz., the fact that these records include a most
remarkable period of drought, and that estimates based upon them are
consequently much more conservative than if based upon any other
records with which I am acquainted.

I may say in the beginning, that I consider the records of the Sudbury River of greater value than those of the other water-sheds of the Boston Water Works, because there is practically no loss from this watershed by the filtration of water through the ground to lower levels, and there is less complication due to public water supplies upon the watershed, diversion of sewage past the lower dam, or by superficial and underground storage of which no account is taken.

Mr. FitzGerald calls attention in his paper to the fact that the Sudbury River records include two years, 1880 and 1883, of most remarkable drought, drier than any other in this vicinity for the sixty years from 1830 to 1890. The period of five years from 1879 to 1883, not only includes these two years, but it also includes three other years, in each of which the flow was below the average. These facts seem to furnish ample justification for Mr. FitzGerald's statement, that "judging by the past, we may feel assured that we have done all that a reasonable care demands, if our works are proportioned to the maximum drought ocurring in so long a period as sixty years."

By making a comparison between the Sudbury and Croton records, we are also confirmed in the opinion that the Sudbury records include a period of most remarkable drought, as will be seen by the following table :

COMPARISON OF SUDBURY AND CROTON RECORDS DURING THE DRIEST PERIODS, VARYING IN LENGTH FROM THREE MONTHS TO SIXTEEN YEARS. *

PERIOD.	Average daily flow for given period in gallons per square mile.	
	Sudbury.	Croton.
3 months	95 000	226 000
8 months	181 000	302 400
1 year	497 000	619 500
5 years	769 000	926 300
16 years	1 079 000	1 057 000

It will be observed from this table that the average flow from the Sudbury River water-shed per square mile during the driest periods of five years or less was very much less than from the Croton, while the average flow of the whole sixteen years was practically the same. It is also true that, taking the whole period in each case, the amount of rainfall was almost exactly the same.

The Croton records would naturally be used in the vicinity of New

* The Sudbury records are based upon the sixteen years from 1875 to 1890 inclusive. The Croton records include the seventeen years from 1870 to 1886, and are taken from a table made by Mr. A. Fteley, Chief Engineer of the New York Aqueduct Commission, and published in the Journal of the New England Water Works Association, for March, 1892.

York in estimating the capacity of water-sheds; but the above compari-
son at once raises the question as to whether water-sheds in the vicinity
of New York are not about as likely to have these severe droughts in
the future as those in the eastern part of Massachusetts; in other
words, was the great drought on the Sudbury water-shed due to its
location, or was it one of those great variations from the average con-
ditions which occur from time to time in all meteorological phenomena
without being assignable to any known law? I am inclined to the
latter view, and consequently believe that estimates of the yield of
water sheds in the vicinity of New York, in order to be on the safe
side, should be based upon the Sudbury rather than upon the Croton
records.

The table presented by Mr. FitzGerald, on page 267, is one by which
the yield of water-sheds can be computed with very little labor, and
with as much accuracy as would result from a tedious calculation
based upon the detailed tables appended to his paper.

A table* similar to his, and also based upon the Sudbury records, is
given below. It differs from Mr. FitzGerald's table, however, in that
his is based upon one square mile of water-shed, including various
percentages of water surface, while mine is based upon a square mile
of land surface with varying areas of water surface in addition.

Daily volume in gallons per square mile of land surface.	Storage required in Gallons per Square Mile of Land Surface to prevent a Deficiency in the Season of Greatest Drought when the Daily Consumption is as indicated in the First Column, with the following Percentages of Water Surfaces.				
	0 per cent.	3 per cent.	6 per cent.	10 per cent.	25 per cent.
100 000	556 000	3 000 000	8 800 000	....	....
150 000	3 400 000	7 100 000	13 400 000	....	....
200 000	9 400 000	11 700 000	18 000 000	....	....
250 000	19 000 000	22 200 000	25 400 000	....	....
300 000	29 800 000	33 000 000	36 100 000	....	....
400 000	52 000 000	54 400 000	57 500 000	....	....
500 000	76 500 000	77 300 000	80 300 000	....	....
600 000	102 000 000	104 600 000	107 100 000	112 800 000	....
700 000	144 400 000	153 000 000	161 600 000	170 700 000	215 900 000
800 000	202 300 000	210 900 000	219 500 000	228 600 000	273 800 000
900 000	346 200 000	349 200 000	352 200 000	353 900 000	381 600 000
1 000 000	514 600 000	516 700 000	519 700 000	523 600 000	532 200 000

The second columns of the two tables are directly comparable
because they represent the case where the water surfaces are absent.
It will be seen that there is some difference in the results, though
perhaps not enough to be of practical importance. The other columns
are not directly comparable, but the comparative results can be shown

* This table was originally published in the annual report of the Massachusetts State
Board of Health for 1890, page 342, and was also published in the *Journal of the Association of
Engineering Societies*, October, 1891, and in the *Journal of the New England Water Works Asso-
ciation*, March, 1892.

by the following example, in which it is assumed that the area of the
water-shed, including water surfaces, is 94 square miles, the area of
water surfaces 6 square miles, and the available storage capacity
9 600 000 000 gallons.

By FitzGerald's Table.

	Square miles.
Area of water-shed, including water surfaces.	94
Area of water surfaces......................	6
Area of land surfaces......................	88
Percentage of water surfaces to total water-shed	6.8

	Gallons.
Available storage...........................	9 600 000 000
Available storage per square mile of water-shed	102 000 000
Average yield per square mile of water-shed, from table............................	565 000
Daily capacity of whole water-shed.........	53 100 000

By Stearns' Table.

	Square miles.
Area of water-shed, including water surfaces.	94
Area of water surfaces......................	6
Area of land surfaces......................	88
Per cent. of water surfaces to land surfaces..	6.8

	Gallons.
Available storage...........................	9 600 000 000
Available storage per square mile of land surface...............................	109 000 000
Average yield per square mile of land surface, from table............................	602 000
Daily capacity of whole water-shed.........	53 000 000

It is sometimes convenient in making preliminary examinations to
remember that ordinary water-sheds where there is no storage, or
where the amount of storage which can be made available is very
limited, as is usually the case with mountain streams, will not furnish
more than 100 000 gallons per day per square mile, while a large pond,
draining a small water-shed, may in some instances furnish as much
as 900 000 gallons per day per square mile of land surface. With the
amount of artificial storage which it is usually feasible to provide upon
water-sheds, the yield is likely to range between 300 000 and 600 000
gallons per day per square mile of land surface.

L. J. LeConte, M. Am. Soc. C. E.—Annual rainfall at and near San Francisco is subject to great variation, and furthermore, there is great difference in rainfall, in the same year, between localities only a few miles apart.

For example, at San Francisco we have as follows:

Inches.

Minimum fall	=	7.4
Maximum fall	=	49.3
Mean annual	=	23.0

At the reservoir sites only 25 miles distant, we have:

Minimum fall	=	20.0
Maximum fall	=	81.0
Mean annual	=	47.0

Generally speaking, the minimum year's rainfall equals one-third the average; the maximum is more than double the average, and more than six times the minimum. Two dry years have come in succession, having only one-half the average rainfall.

The flow of streams is correspondingly irregular. They flow from December to May, and are dry the rest of the year. During dry years there is no flow and sometimes no fresh supply enters the reservoirs for a period of six hundred days. Meanwhile evaporation from water surfaces equal 60 inches per year. Hence the practice in California, as to storage reservoir, is quite different from that in the Eastern States, where streams are running, more or less, the whole year.

Large reservoirs are a necessity with us and the aim is always to catch all the flow that the water-shed furnishes, nothing is allowed to go to waste, as a rule. Reservoirs are generally designed on the basis of 85 million cubic feet of storage capacity for each square mile of water-shed. This is four to five times as much as most cities have adopted.

The "catch" or amount collected each year is also subject to still greater variation, but as an average may be estimated at 30 per cent. of rainfall. During years of light rainfall, even up to 20 inches, the catch is practically nothing. Where the variation is so great, it is impossible to make a catchment table which would command confidence, but if we take a wide grasp of the whole subject, I think we are justified in assuming the following approximations:

RAINFALL.	CATCH.
10 inches.	0.5 inches.
20 "	2.0 "
30 "	9.0 "
40 "	18.0 "
50 "	30.0 "

Mr. Stearns' statement that it is impracticable to secure more than 600 000 gallons per day per square mile of water-shed, is a very important contribution to our list of useful facts. At San Francisco similar observations have been made, and the final conclusion arrived at is, that a reliable daily supply of 1 000 000 gallons will require from 4 to 4½ square miles of water-shed. This is a result of fourteen years' experience (1877 to 1891), the mean annual rainfall being about 47 inches. The above catchment table throws ample light upon the true cause of this difference.

E. SHERMAN GOULD, M. Am. Soc. C. E.—An important feature of this paper is its bearing upon the capacity of spill-ways, or overflows of reservoirs. The author lays down as guiding data, that, in districts analogous to those treated of, 1 square mile of drainage area furnishes approximately 1.5 cubic feet per second yearly average, which volume may be increased a hundredfold, or up to 150 cubic feet per second, for twenty-four hours, by maximum freshets. This maximum flow is certainly very large, being 3.4 times greater than that of the Sudbury water-shed in February, 1886, mentioned by the author. It will be interesting to make some calculations, to see to what dimensions of spill-way these data lead us.

I have stated elsewhere that, in want of a better, the following formula gives fairly well the proper average length in feet of spill-ways in terms of the drainage area expressed in square miles :

$$L = 20 \sqrt{A}.$$

I will use this length in the following calculations, determining the vertical dimension H, by means of the well-known formula for discharge over weirs :

$$Q = 3.2 \times L \times \sqrt{H^3},$$

in which $Q =$ cubic feet per second, $L =$ length of weir in feet, and $H =$ height in feet from sill to surface of smooth water. The coefficient 3.2 is, of course, subject to variation ; the above value is a near enough average for the present purpose.

If, now, we calculated the values of L and H by means of these formulas for a series of reservoirs impounding the flow from water-sheds respectively of 9, 25, 100 and 400 square miles, yielding 150 cubic feet per square mile per second, we shall have :

Area, 9 square miles,	$L = 60$,	$H = 3.63$
" 25 " "	$L = 100$,	$H = 5.16$
" 100 " "	$L = 200$,	$H = 8.19$
" 400 " "	$L = 400$,	$H = 13.00$

These values of H represent the height of the cross-section of water going over the sill of the dam. The crest of the dam must be raised still higher, the additional height depending upon the nature of the

dam, and the probable height of the waves which may be pro-
duced behind it.   It is evident that the above figures call for spill-
ways of great capacity.   It would be very interesting to know if the
results obtained agree with the practice of our leading hydraulic
engineers.

Some years ago I made calculations for the proper height of spill-
way corresponding to a maximum flow of 40 000 000 gallons per square
mile per twenty-four hours, or about 62 cubic feet per second, this
being, as far as I could ascertain, about the maximum observed at the
Croton dam.   I found, using the previously given formula for length,
that the proper height, in feet, was the cubic root of the number of
square miles in the water-shed.

Using this rule for the same areas as those previously taken, we
have :

$$\text{Area, 9 square miles,} \quad L = 60, \quad H = 2.08$$
$$\text{`` 25 ``} \qquad\qquad\text{``} \quad L = 100, \quad H = 2.92$$
$$\text{`` 100 ``} \qquad\quad\text{``} \quad L = 200, \quad H = 4.62$$
$$\text{`` 400 ``} \qquad\quad\text{``} \quad L = 400, \quad H = 73.7.$$

For a maximum discharge of 62 cubic feet per second per square
mile, we should then have the two general formulas for length and
depth of spill-way :

$$L = 20\sqrt{A}\dots\dots\dots\dots\dots\dots\dots\dots\dots (1)$$
$$D = \sqrt[3]{A} + C\dots\dots\dots\dots\dots\dots\dots\dots (2)$$

in which L = length in feet ; D = total height in feet from sill to
crest ; A = drainage area in square miles, and C = a varying height from
high water-mark to crest of dam depending upon circumstances already
mentioned.

To adapt (2) to a flow of 150 cubic feet per second per square mile,
it will suffice to multiply $\sqrt[3]{A}$ by 1.77.

Such general formulas cannot be taken as absolute guides, but may
be useful in affording a rational foundation for more special study in
particular cases, the vital question always being: what is the maximum
flow per second which can possibly occur under the most unfavorable
combination of circumstances ?   Now, any engineering problem con-
sidered in this light, i. e., that of the most unfavorable conditions
conceivable, naturally leads to a very expensive solution, and it is,
therefore, probable that the tendency will always be in dam build-
ing, to restrict the dimensions of the overflow below what the more
pessimistic view of the case would dictate.   In masonry dams built
across rocky ravines, we are comparatively indifferent to the possibility
of the dam being overtopped by a freshet, but in earthen dams this
constitutes, perhaps, the greatest peril with which they are menaced.
It is for this reason, among others, that my opinion remains un-
shaken that such dams should always be provided with a substantial

center wall of hydraulic masonry, carried well above the sill or lip of
the dam, and the rip-rapping of the inner slope carried still higher.
As a rule we may expect that the heavier the freshet, the shorter its
duration; so that if a dam thus protected should be overtopped, we
might expect the flood to subside before it should have entirely
stripped the outer slope, and thus caused the center wall to yield,
particularly if liberal discharge culvert capacity is also provided and
made use of.   And even if ·the worst should occur, and the center-
wall be breached, time, that priceless element in such cases, would be
gained, and the catastrophe greatly modified.   For the only difference
between the harmless emptying of a reservoir, and a Johnstown
disaster, is one of time.

As our profession, like that of the law, depends in its exercise
largely upon precedent, it is greatly to be desired that statistics
be gathered respecting the dimensions of the overflow of existing
reservoirs, together with their corresponding water-sheds and meteor-
ology.

If the proper depth of spill-way for discharge of 62 cubic feet per
square mile per second be $\sqrt[3]{A}$, the proper depth for any other quantity,
Q, may be obtained by multiplying $\sqrt[3]{A}$ by the coefficient C, derived
from the relation, $\dfrac{Q}{62} = \sqrt{C^3}$, which reduces, using round numbers, to

$$C = \frac{\sqrt[3]{Q^2}}{16}.$$

We would then have the two general approximate, or rather tenta-
tive, formulas for length and depth of spill-way to discharge a given
number Q of cubic feet per square mile per second, replacing those
before given:

(1)..........................$L = 20 \sqrt{A}$ (as before given.)

(2) ....$D = \dfrac{\sqrt[3]{Q^2}}{16} \times \sqrt[3]{A} + C.$

MANSFIELD MERRIMAN, M. Am. Soc. C. E.—The reason why a rain-
gauge on the top of a building indicates a less amount of rainfall than
one on the surface of the ground, has not yet been made clear.   It is
generally assumed that the lower gauge gives the correct result, and
that the indications of higher gauges are unreliable.   Mr. FitzGerald
asserts that the signal service observations of rainfall, made on the tops
of high buildings, are untrustworthy.   My own view of this matter is
that the higher gauge gives the more reliable result, particularly in
cities.   This opinion is not founded on any experimental evidence, and
therefore cannot be regarded as of much value; but as I have not seen
the theory stated by any writer, it may be worth while to note it here
and to give the reasons in favor of it.

The size of a rain-drop, it seems to me, is not constant, but increases

as it descends.   This is due to the evaporation which constantly goes on, even during a rain and which is a maximum near the surface of the ground where the drops of rain are broken up by the impact on trees, grass and earth.   Consequently, the air near the surface of the ground becomes saturated with moisture, and this is partly taken up by other drops, whose size accordingly increases as they fall.   A rain-gauge on the surface of the ground hence measures too great a quantity, for a part of the water which falls into it has fallen before on surrounding objects.

If there is anything in this theory it would be expected that the difference in the readings of two gauges, one low and the other high, would vary with the humidity of the air and also with the force of the wind. A series of records with several self-registering rain-gauges will enable a discussion to be made which will throw some light on a question now in much doubt.

In a large city I can conceive of no better place for a rain-gauge than on the top of a building.   No one would advocate putting it in an alley or court yard, and a public square filled with trees does not seem a good place.   By locating it on the roof of a building the influence of heated pavements and walls is largely eliminated, and these would probably have more effect on the evaporation of water that has fallen than would be the case in a small town or in the country. Out on the treeless prairie it seems probable that the height of a rain-gauge is of less importance than elsewhere.

The only definite reason why a rain-gauge should be located at the surface of the ground, which I have been able to find, is that at a higher elevation the drops of rain "move in parabolic curves" whenever there is wind.   This reason, however, explains nothing, for, whether the path of the drops be vertical, or inclined, or curved, if their size remain the same, the same quantity of water passes through two horizontal planes of different elevations.   As numerous observations show that this is not the case, I conclude that the size of a rain-drop generally increase as it descends.

EDMUND B. WESTON, M. Am. Soc. C. E.—I have been very much interested in the reading of Mr. FitzGerald's paper, more especially as during the past twelve years, at different periods, I have spent considerable time in examining and comparing data relative to the flow of water from drainage areas.   I do not know of any work of this nature that has been done more carefully than the work upon the Boston Water Works, and the manner in which the results have been presented by Mr. FitzGerald seems to be particularly valuable and instructive.

The following table, giving results from a number of European water-sheds, which I have compiled from some data that I have at hand, may be of interest for comparison:

TABLE SHOWING THE FLOW OF WATER FROM A NUMBER OF EUROPEAN
WATER-SHEDS.

RIVERS, ETC.	Average annual rainfall in inches.	Mean discharge per square mile.	Per cent. of rainfall run off.	Drainage area in square miles.	Years in which observations were made.
River Lee, Hertfordshire, England....................	28.75	30.5	24	444.	1851 1852 1856
Loch Katrine, District Scotland......................	103.30	362.0	79	71.6	1854
Loch Lubnaig, District Scotland.....................	66.70	225.7	76	69.7	1847
River Bann, and Lough Neagh, Ireland.............	25.92	94.8	78	2205.0	1856
Brosna, Ferbane, River, Ireland.....................	35.23	99.0	64	446.	1852 1856
River Robe, Mayo, Ireland...	49.35	128.9	60	109.4	1851 1852
River Saône, France..........	*32.6	86.1	61	11551.0	1852 to 1855, inclusive.

* Rainfall average of twelve stations.

I quite agree with Mr. FitzGerald in regard to the unreliability of some of the earlier records of rainfall. I can recall, among others, a series of observations covering nearly half a century, that were recorded by a man celebrated for his scientific attainments. These records, that were concluded in 1876 owing to his decease, have many times been used and quoted as examples of the early rainfalls. The gauge used for making the observations was located on the top of a fence about 6 feet above the ground, and the snow recorded as rainfall was simply the amount, melted, that was caught in the gauge. Experiments covering a number of years, that were made under my direction, show that at this elevation a gauge collects much less rain than one located about 1 foot above the ground, also that more or less of the snow that enters a gauge is not unfrequently blown out before it can be melted and measured. The results that I obtained indicate that in a gauge located 1 foot above the ground, and the snow measured at the surface of the ground (by cutting out and melting a section) the average annual rainfall will measure at least 13 per cent. more than in a gauge located and used as was the one which I have just mentioned.

B. W. DeCOURCY, M. Am. Soc. C. E.—Mr. FitzGerald's interesting tables of rainfall induce me to write the following on the rainfall and phenomena arising therefrom in Washington and Victoria, B. C. I send some extracts from the records of the different stations kindly furnished me by the observers in this region. They will be found to

vary much in a small distance, only a few miles in position making a material change.

It is necessary to state that Mr. FitzGerald's remarks as to the accuracy of observations will obtain here much more than with the region he treats of, as many of the observers are volunteers, whom the
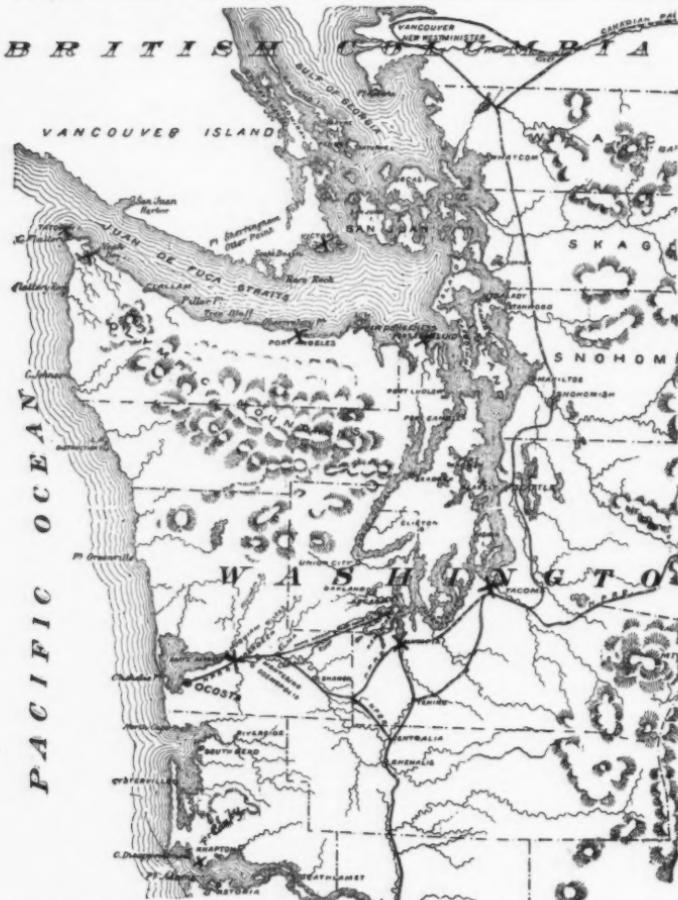
love of science has induced to devote some time from their other pursuits to this interesting subject. Still, the rainfall here is very great, and appears to be influenced by different causes.

Beginning on the foot hills of the Cascade Range of mountains,

there is a maximum fall, which decreases until those of the Olympic are reached; but thence there is an increase until the maximum is again reached at Neah Bay or the capes, at the entrance to the Straits of Juan de Fuca, and this maximum prevails along the coast and west of the Olympic Mountains.

This entire region is covered with a dense growth of timber, consisting of the Douglas fir, spruce, cedar and *tsuga-merteusiana* (commonly called hemlock, though quite a different timber from the hemlock of Canada and the east); besides, there is a dense growth underneath of sallal, vine maple and small woods, so completely impervious to wind and the solar rays, that there is scarcely any evaporation, and this, and the fallen timber hold back the rainfall from the streams and deliver it so gradually, that pronounced freshets are quite uncommon in the large or comparatively large streams.

There are no very large streams in this State west of the Cascade Range, and where freshets do occur, thay are caused by the melting snow. However, the entire region is cut up with small creeks, so that a 40-acre tract will be found watered with numerous small branches.

I have paid some attention to the rainfall in the valley of the Chehalis (one of our largest streams), which discharges into Gray's Harbor, and has a water-shed of about 3 000 square miles. The results show the smallness of the evaporation. When Chief Engineer of the State Harbor Line Survey of the different harbors forming the estuary of the Chehalis, called Gray's Harbor, I gauged the river and measured it accurately in several places.

Aberdeen, Hogmain and Ocosta show a rainfall of 90.82 inches, which is about that of the entire valley.

The rainfall is 4 689 767 064 934 gallons. At the river's mouth the average current is, when running out at low tide, 5 feet per second.

RAINFALL, 1891.

	NEAH BAY.	PORT ANGELES.	TACOMA.	ABERDEEN.	ABERDEEN, 1892.
January	15.93	2.96	5.38	......	5.75
February	6.64	.99	2.68	......	4.95
March	9.80	2.43	2.75	4.57	......
April	11.84	2.49	4.91	9.23	......
May	.91	1.53	1.92	2.75	......
June	6.17	.94	2.93	4.92	......
July	2.60	.00	0.05	1.21	......
August	2.11	1.58	2.76	2.02	......
September	10.78	2.35	4.17	8.52	......
October	10.06	3.33	5.15	6.69	......
November	23.06	3.20	7.63	19.96	......
December	23.91	5.78	10.55	19.34	......
	......	......	......	10.70	For 2 months, 1892.
	123.81	27.58	50.88	89.91	......

The sectional area is 26 400 square feet. The prevailing winds in winter are southwesters; in summer, those from the south and south-west bring rain, and northwest and north or easterly, clear and fine weather.

Neah Bay, average for eight years........ .... 104.
Port Angeles, average for eight years............ 28.91
Tacoma, average for five years................... 42.84

	Average annual rainfall.
Dayton, Washington.............................	27.75
Fort Canby, Washington.........................	45.71
Olympia...	59.72
Tatoosh...	75.18
Port Townsend..................................	17.00
Portland, Oregon...............................	54.64

Monthly and Annual Rainfall, Victoria, B. C.

In Inches. Ten Years—1881 to 1890.

	1881.	1882.	1883.	1884.	1885.	1 886.	1887.	1888.	1889.	1890.	Mean.
January............	3.84	2.28	5.67	5.25	9.15	3.09	6.58	5.02	2.84	3.54	4.72
February..........	8.84	3.55	3.26	2.11	3.84	3.17	4.82	1.77	1.12	2.33	3.48
March.............	1.57	4.02	1.56	0.38	0.32	2.94	5.36	3.53	1.50	1.50	2.27
April.............	2.70	1.24	2.02	1.02	0.53	1.67	0.76	2.26	1.83	0.86	1.49
May...............	1.48	0.53	0.74	0.73	1.30	0.45	1.32	0.19	1.01	0.98	0.87
June..............	1.57	0.42	0.53	1.59	0.25	1.00	0.48	2.23	0.77	2.10	1.09
July..............	0.90	1.24	0.06	0.48	0.06	0.80	0.27	0.34	0.00	0.64	0.48
August............	0.79	0.99	0.00	1.84	0.02	0 73	0.01	0.42	1.04	0.12	0.59
September.........	0.82	0.59	1.65	1.66	4.00	1.59	1.16	1.01	2.33	0.33	1.51
October...........	4.11	4.30	1.58	4.88	2.73	2.32	2.75	3.35	2.08	7 52	3.56
November..........	5.25	3.32	6.03	1.60	3.47	1.92	5.36	3.69	1.76	1.74	3.41
December..........	6.13	5.37	4.55	1.95	2.47	7.16	9.18	1.96	2.28	8.28	4.93
	37.99	27.85	27.65	23.49	28.14	26.84	38.05	25.77	18.56	29.94	28.41

I am under obligations for records to Mr. C. P. Culver, attorney at law, voluntary observer, at Tacoma; Mr. Irvine, at Port Angeles; Mr. Charles Addie, at Neah Bay; and Mr. Mack, at Aberdeen.   Also to Mr. A. L. Going, C. E., at Victoria.

I am indebted to R. R. Ball, Captain and Assistant Surgeon U. S. A., for the record of rainfall at Port Townsend, Washington, as given below.   Mr. Ball is Surgeon of the Post.

Monthly rainfall at Port Townsend, Washington, from July 1st, 1891, to June 30th, 1892.				Monthly average rainfall and melted snow for past ten years.	
1891.	Inches.	1892.	Inches.	Year.	Inches.
July	.38	January.........	1.35	1882	2.02
August...............	2.52	February..	1.78	1883	1.68
September	1.78	March..............	1.86	1884	1.63
October..............	1.12	April...............	2.42	1885	1.38
November............	3.30	May................	2.90	1886	1.83
December............	5.34	June.....	.37	1887	1.56
				1888	2.03
				1889	1.25
	25.12 inches total for year.			1890	1.66
				1891	2.08

LEWIS M. HAUPT, M. Am. Soc. C. E.—The subject that Mr. Francis has touched upon is one that is of national importance. I think this is a question which we should take a little time to consider in consequence of its humanitarian aspects. We want to do as much as we can to alleviate distress from floods.

I have recently prepared a paper on the subject of the Mississippi problem for the *Engineering Magazine*, expecting to bring the question up for discussion at another time; but as it seems to me to be germane here, with the consent of the Society, I will give a few abstracts from it.

" In the voluminous discussions and hearings upon this oft-mooted subject, no one has ever attempted to show that the amount of sediment ejected from the river's mouth at all approximates to that received from the tributaries or eroded from its banks and bed, yet it is a self-evident proposition that, unless these two quantities be equal, there must be an annual and permanent deposition in the bed of the stream, causing that bed, taken in toto, to rise. This opinion has been feelingly denied, and it is even attempted to prove that the beds of the far-famed Po, Danube, Yellow, Ganges and other alluvial streams of lesser extent are not rising.

" In a state of nature the river has provided its own 'dump' by overflowing the wide valley through which it flows, dropping its heaviest sediment nearest its edges, and building up a traverse highest near the brink, sloping away from the borders to the lower swamps and bayous beyond. Thus the rich alluvial territory from Cairo to the Gulf has been constructed out of the terrane from the mountains, and an ample dump has been furnished for the silt-bearing stream; but if the river be leveed along its entire course, and be compelled to bear its burden over these thousand miles, it will not be found able to cut deeper when surcharged with sediment, but it will drop its load in every available pool or bend at high water, and assuredly and rapidly raise its bed.

" The remedy suggested by a Southern author is to open up the lateral streams and divert some of the flood waters through more direct, shorter and steeper channels to the Gulf, the same as has been so frequently urged by Captain Cowden under the name of the 'outlet' system, and which has been so vigorously opposed by many engineers em-

ployed on the river, because of the alleged formation of bars below the outlets.   This feature, however, is even now being carefully studied by some members of the corps of engineers in charge of sections of the river, and the data thus far collected would seem to show very grave doubts upon the supposed injury to the lower reaches from the outlets.

" Following Nature's method, however, and compromising with the land owner, it would seem practicable to provide a sufficient number of large lateral subsiding basins or lakes by enclosing extensive areas at intervals where the topography admitted of economical construction, into which the flood waters could escape, and in which, the velocity being reduced, a large part of the silt would be precipitated, while, after the passage of the crest of the flood-wave, the clearer waters from the reservoirs would return to the river and become useful for naviga-tion.   In short, instead of serving the sole purpose of maintaining the water supply for low stages, as do the reservoirs in the upper Missis-sippi, they would also reduce the rate of raising the bed by providing lateral dumping grounds outside of the bed of the stream, and would also reduce the flood plane and dangers from inundations.   There are numerous places on the river where, by making return dikes extending back to the bluffs, many square miles of land, now of little value, might be utilized for such safety valves for the river, with substantial benefit and at a comparatively small cost.

" With reference to the probable effects of the lateral or outlet plan, while there is no grand precedent for it, there are many instances which would point strongly to its success, and one of the best which has re-cently fallen under my observation is to be found in the lowering of the flood plane of the Tyne, in England, due to the removal of bars and obstructions at its mouth and along its lower reaches as far as Newcastle.

" Captain Cowden has reduced his outlet theory to the aphorism ' make the overflow greater than the inflow and there can be no over-flow.'   In applying it he proposes to cut off a portion of the inflow by conducting it through lateral channels to the Gulf and thus preventing the tributary sediment from ever entering the main stream.   This, cer-tainly, is a desirable feature which appears to have been overlooked.

" It does not appear rational to expect any permanent improvement to a stream until the obstructions at its mouth be reduced and the sur-face slope in its lower reaches be increased, whether these obstructions be in the nature of mud or of water fed by tributary courses.   The systematic improvement of the river must treat the problem as a whole, and provide at high stages for the rapid emptying of its basin at the outlet, the retardation of its filling by its tributaries, provision for deposition of its load and temporary escape of its excess of water along its course; while for the low water stages for navigation the stream must be canalized so far as practicable to retain a nearly uniform velocity."

B. M. Harrod, M. Am. Soc. C. E.—I want to say a few words, not in discussion of Mr. FitzGerald's interesting paper; but in reply to re-marks relative to the Mississippi River, made part of that discussion by Professor Haupt. In these remarks, as I understand them, there was suggestion made of relief from great floods by the diversion of tribu-taries and by the use for storage reservoirs, of the great basins which lie on either side of the river.

The further diversion of any of the tributaries presents insuperable

difficulties. The Red River is already diverted from the Mississippi, about 6 miles from its mouth, into and through the Atchafalaya. The flood discharge of the Red River is about 200 000 cubic feet per second, while the Atchafalaya has a flood discharge, approximating twice that quantity. The latter stream therefore serves not only to divert the Red, but also, in emergency, as an outlet to the Mississippi. With this exception, the diversion of any one of the tributaries of the Mississippi River is impracticable.

To divert any other of them would involve a cut hundreds of feet deep, hundreds of feet wide, and hundreds of miles long, or an undertaking greater than the construction of the Suez Canal. An examination of the topography of the valley will establish this.

Regarding the use of the great alluvial basins as reservoirs for surplus flood water, there are difficulties of a different character. The lands in these basins are owned, under valid titles, by states, corporations and individuals. Many of them, which would be submerged in a reservoir, have much value, and are in successful cultivation. It is certainly true that all these lands are steadily appreciating. The Yazoo Basin is now substantially reclaimed from overflow, and the value of the lands therein has, in many cases, if not generally, increased 500 per cent., and homes and occupation have been given to a steadily increasing industrial population. The owners have in view, therefore, plans other than this abandonment as overflow reservoirs. They propose to complete their reclamation.

But I want more particularly to speak of the engineering objections to such a project. It is not established that general relief is found by using these basins as reservoirs to receive and hold back for a time, the water overflowing into them from great floods. An examination of the gauge record shows that, at the mouths of the tributaries draining these basins back into the Mississippi, the highest readings have been reached when a great overflow was returned to the main trunk, and that while the improvement or completion of a levee system has, for a first effect, the increase of flood heights along those parts of the fronts of the basin where overflow had previously occurred, yet that the transmission of the entire flood between levees has not had a corresponding effect in the gauge heights at the mouth of the tributaries. The use of the basins as reservoirs does not, therefore, serve as a general relief from excessive flood heights, and it certainly would not from overflow.

I think that the main idea underlying Professor Haupt's discussion is that the bed of the Mississippi River is rising. Observations bearing upon this point have been systematically made for the past thirteen years, and a few low water gauge records extend back about three times that period. It is, of course, impossible that this question of an elevation or depression of bed can be settled in so brief a time; but it is safe to say that, as far as these observations go, they

afford no evidence of any general or progressive rise of the bed of the river. They do indicate that below crevasses or outlets there is a loss of depth and section, and that the closing of these, by rebuilding the levees, tends to restore the previous capacity of the bed.

Other facts seem to connect the existing elevation of the bed of the river with the volume held within the banks. It has always been observed that the range between high and low water is different in different parts of the river. It is greatest at or about the mouth of a tributary when its natural discharge, together with the return of over-flow into the basin, is added to the volume of the main river. It is less along the front of the basins, where the discharge is decreased by loss of volume over the banks. In general figures the flood discharge and the range from low to high water are both one-fourth greater at the mouths of the tributaries than they are at intermediate points along the front of the basin, or 43.5 and 35.3 respectively. In plotting the low and high water slopes, it is found that the high water line is quite regular when compared with that of low water, which has depressions where the flood volume is greatest and elevations where the flood volume is less. In other words, the irregularity of the low water line, which generally conforms to the shape of the bed, follows the depressions of the bed where there is greater, and the elevation of bed where there is the less flood discharge. This relation of elevation of bed to flood volume was among the earliest observations. There is no evidence that it is progressive. On the other hand, there is no evidence that the maintenance of a more nearly uniform flood discharge through-out the length of the river by levees has yet caused any greater uni-formity of bed level, although it is reasonable to expect such a result.

DESMOND FITZGERALD, M. Am. Soc. C. E.—It is not entirely correct to say that no account has been taken in my tables of the storage in the ground, for the reason that more or less of this action has entered into the measurements of the Sudbury River in the past sixteen years, due to the drawing down of the reservoirs and Whitehall Pond during the summers. It was not from lack of some information on this subject, however, that I decided not to make a separate account of it in the tables. The soils surrounding reservoir sites differ so much in character and the situations of the reservoirs themselves affect this question to such a degree that it seems to me best, in the lack of ade-quate data, to leave any such allowance to the judgment of the engineer rather than to attempt to complicate an already complicated problem with such a variable and uncertain factor.

The following experiment was made under my direction to determine exactly what additional water was received from the storage in the ground in the case of Basin No. 4 of the Boston Water Works, situated on the Sudbury water-shed.

This reservoir of 162 acres has a surveyed water-shed of 6.434 square miles, including the reservoir. Its situation is high rather

than low, and the geographical formation is unmodified drift. Its water comes almost entirely from a feeder at one end of the basin and so is favorably situated for the experiment. A weir 20 feet wide was erected at the inlet, with a smaller weir for measuring summer flows, and a self-recording gauge registered continuously the amount of water entering the reservoir. The area of the water-shed above the river was 5.466 square miles, including the reservoir, which left only 0.968 square mile of contributing water-shed around the margins unmeasured. It was assumed that this area gave a *pro rata* supply. Rain and evaporation gauges were erected, the latter a floating gauge properly exposed. It will be impossible in the limits of this discussion to give the details of the experiment, which covered a period of six months, during which the reservoir was drawn down 12 feet. The following table, however, shows the results of the measurements and estimates:

EXPERIMENT TO DETERMINE THE YIELD OF THE WATER TABLES SURROUNDING BASIN 4.

Date of Periods.	July 1–16.	July 16–Sept. 27.	Sept. 27–Oct. 7.	Oct. 7–Jan. 1.
Grades.....................	214.81 to 214.84	214.84 to 202.91	202.91 to 202.93	202.93 to 207.85
Days.......................	15	73	10	86
Flow over weir—gallons..	9 952 000	52 808 000	2 618 000	190 950 000
Flow from land below weir	1 287 000	7 062 000	366 000	26 515 000
Rainfall on water surface.	4 510 000	38 800 000	516 000	39 825 000
Water from water tables..	6 617 000	66 826 000	2 864 000	....
Drawn through gates.....	5 960 000	704 920 000	....	....
Leakage (measured)	1 886 000	7 696 000	817 000	7 866 000
Evaporation...............	12 870 000	43 350 000	4 687 000	22 365 000
Contributed to water tables........	....	....	....	3 099 000
Storage—gained or lost..	+1 650 000	−590 470 000	+860 000	+223 960 000

From this experiment it was assumed that if the water had been kept at a uniform level, there would have been a uniform subterranean flow into the basin of 300 000 gallons per day, derived from rain percolating through the ground; and with this allowance the water tables contributed to the storage capacity of the reservoir 44 900 000 gallons, while it was falling 12 feet. If these figures are correct, and no expense was spared to get at the truth, then the storage capacity of the reservoir to the 12-foot contour, viz., 590 470 000 gallons, received less than 8 per cent. additional supply from the water tables. It is obvious, however, that this figure would be very much larger in the case of a reservoir situated in low gravelly ground; and, on the other hand, less in a rock formation.

Admitting that there is a considerable accession to the storage from the water tables surrounding a reservoir, the question now arises, will it not be better, save perhaps in exceptional cases, to ignore this storage in order to be on the safe side.

The sixteen years during which the Sudbury has been measured contained a remarkably dry period, as I have already pointed out; and in this lies the great value of the results contained in the table and dia-

gram for storage capacity ; but is it not presumptuous to suppose that no greater period of drought will occur in the future ? And is it not better to have the additional help of the water tables against such a contingency, rather than to cut down the capacity of the storage as outlined in the paper under discussion? Again, it is always wise to have some water left in the bottoms of our storage reservoirs for sanitary reasons.

Mr. Stearns has shown so well the agreement of our figures, although made on a different basis, that I will not dwell on this point.

Having found from topographical surveys and careful studies of the Sudbury River Water-Shed, that it will be possible to secure a total storage of 19 671 000 000 gallons on 75.199 square miles of water-shed, or 262 000 000 gallons storage per square mile, with 9 per cent. water surface, and providing in time of greatest drought a daily draft of 785 000 gallons per square mile, with a total daily draft of 59 000 000 gallons from the 75.199 square miles, and at a reasonable cost, I am led to believe that it will be found practicable and wise in some cases to develop a water-shed to this amount, but I cannot agree with Mr. Herschel, when he calls this a moderate development of a water-shed.

Professor Merriman will find in the literature on the subject, published in England, very full discussions and theories to account for the fact, which is undisputed in that country among experts, that the records of elevated gauges are untrustworthy. To satisfy myself in regard to the matter, I once had a series of towers built up to 60 feet in height, on which were placed rain gauges specially designed for the purpose of accurate observations, and which have been favorably mentioned in the reports of the Signal Bureau. There were nine gauges, from the surface upwards, placed so as not to interfere with each other, and daily observations were taken for three years, together with the velocity of the wind. The results convinced me that there was no doubt about the inaccuracy of the elevated gauges. The total rainfall collected at the height of 60 feet was about 82 per cent. of that collected at the ground level. It does not require these facts, however, to convince us of the accuracy of the statement made at the bottom of page 254 of my paper. It is only necessary to compare the observations made on the tops of high buildings with the observations made in gauges properly exposed on the ground in the same region, and which agree very well, even when several miles intervene, to become satisfied on this point. I believe the Signal Service officers will also endorse my statements. The wind is probably at the root of the trouble.

Mr. Gould's discussion in regard to spill-ways is interesting, but I suggest that, in the majority of situations in which water-works dams are situated, it is wrong to depend upon the core wall to save an earthen dam in times of freshet; and, after all, would it not be cheaper, considered merely as a matter of dollars and cents, to make the spill-way of ample capacity, rather than to shorten it, and repair the dam whenever it is overtopped ?

TRANSACTIONS.

548.

TEST OF POWER REQUIRED TO DRIVE ELECTRIC STREET CARS, AND TOTAL EFFICIENCY OF MOTOR.*

By Louis B. Bonnett, Jun. Am. Soc. C. E.

READ SEPTEMBER 7TH, 1892.

The tests described in this paper were made by the writer and Mr. Wallace M. Hill, on the Sea Shore Electric Railway at Asbury Park, N. J., in the summer of 1889, the object being to obtain a measure of the power taken from the wires to drive a car under average conditions of service, and in the efficiency test to obtain the total efficiency of the motor, that is, inclusive of friction of shafts, gearing and axles. The figures obtained were the ratio of the power delivered at the circumference of the driving wheels to that taken from the wires, and represent the commercial efficiency of the car.

The instruments used in making the car tests were, an Ayrton and Perry ammeter, and a Weston direct reading voltmeter; in addition to

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* Discussions on this paper will be received until November 15th, 1892, and published at a later date.

these, in the efficiency tests, two spring balances of 400 and 120 pounds capacity, respectively, which formed part of a differential Prony brake and a speed counter, were used. The ammeter and voltmeter had been carefully standardized just previous to the tests, and the spring balances tested.

The road was built by the Daft Electric Co., and was of the overhead trolley system, having a double conductor giving a complete metallic circuit, the main and return wires being suspended on poles on either side of the street and connected with the trolley wires in the center at each pole. The track was in good condition except where covered with sand and clay from the streets; average grade about 1 per cent.; minimum curve radius, 50 feet. The line is about $3\frac{1}{2}$ miles in circuit, single track with turnouts, and the schedule speed including stops 8 miles per hour.

The car on which the tests were made was of the open summer pattern, weighing complete about 6 600 pounds. The motors were of the Gramme type, designed for 220 volts pressure, series wound and rated by the builders at 20 horse-power. The frame of the motor truck consisted of two $\mathbf{I}$ bars bent in the shape of a fork or $\mathbf{U}$, the forward ends of which supported the driving axle, the motor and gearing being supported between the body of the $\mathbf{U}$. These $\mathbf{I}$ bars are joined at the base of the $\mathbf{U}$, and from their junction a bar projects which has at its end a journal, carried in a swivel held between two collars on the center of the follower axle, this arrangement giving considerable flexibility to the frame. These motors were provided with carbon brushes permitting rotation of the armature in either direction without change. The field was wound with three sets of windings, primary, secondary and tertiary, variations of power being obtained by making connection with one, two, or all three of these coils.

Connection was made from the car to the overhead conductors by means of a small four-wheeled traveler or trolley, of which the two wheels on one side run on the positive and those on the other on the negative wire, the two sides being insulated from each other. From this trolley two flexible conductors covered with rubber tubing ran down to the opposite ends of an insulated holder or mallet, which could be inserted at will between two metallic clips on the hood of the front platform of the car. From these clips two wires ran around the top of the hood, down under the floor, forward, and up to a switch on the

platform. The lever of this switch box could be placed at will at "off" or first, second or third notch, forward or back.

In making the car tests the voltmeter was connected in shunt circuit by two fine wires to the main wires on opposite sides of the two clips mentioned. The ammeter was put in direct circuit by cutting one of the main wires where it entered the car ; the instruments were placed on one of the front seats of the car where the observers could read them at will. The figures for the electric horse-power taken from the wires given in the following tables were obtained by the formula

$$\frac{C.E.}{746} = \text{H.-P., in which}$$

C = Current in ampères

E = Electromotive force in volts, and

746 = Constant.

The results of the tests are given in the following table:

TABLE No. 1.

Number of test.	Readings taken.	Mean Electric horse-power.	Passengers, average.	REMARKS.
3	41	7.06	10	{ Run most on 3d notch; motor new and { stiff; special trip.
4	44	7.18	10	{ Run most on 3d notch; motor new and { stiff; special trip.
5	52	7.20	10	{ Run most on 3d notch; motor new and { stiff; special trip.
6	32	5.96	6	{ Run on 2d and 1st notch; car in regular { service; readings spaced to give average of { regular trips.
7	29	7.70	7	{ Run most on 1st notch; readings taken for { high power, exerted on starts and curves.
8	24	8.62	6	{ Run most on 1st notch; readings taken for { high power, exerted on starts and curves.

All of these tests were made on the same car, and each represents the average of readings on one round trip; the first three in the table were made on the first trips with the new motor with which all the cars on the line were being fitted, and as the journals and gearing were new and stiff, gave a higher figure for the power used than in test 6, which was the best one made. In tests 7 and 8, more attention was paid to getting readings of maximum power taken at starting, and while rounding the sharp curves, on which the tracks were often covered with sand, consequently their average figures out higher than it should for a fair test over the whole circuit.

The full record of test No. 6 is given in Table No. 2; in this test the readings were taken as nearly as possible at equal intervals, except 15, 27, 30 and 36, which show the large amount of power exerted to start the car (which, with the passengers, weighed about 7 500 pounds) into full speed in about its own length, on a sandy track.

TABLE No. 2.

	Volts.	Ampères.	Horse-power.	REMARKS.
1	212	41.7	11.85	Curve (1).
2	216	11.6	3.35	" "
3	208	13.9	3.87	" (2).
4	208	25.5	7.10	" "
5	212	11.6	3.32	Level.
6	212	18.5	5.25	Up grade, 1st notch.
7	216	13.9	3.97	Down grade, 1st notch.
8	216	7.	2.02	" "
9	216	9.3	2.73	Up grade, "
10	192	9.3	2.39	On bridge, 2d notch.
11	208	20.8	5.79	Level, faster, "
12	208	18.5	5.18	" " "
13	208	23.2	6.46	Curve (3), "
14	200	27.8	7.45	" " "
15	184	97.2	*23.84	Start, on turnout (1), 1st notch.
16	216	18.5	5.35	Level, 2d notch.
17	212	20.8	5.91	" "
18	204	32.4	8.86	Curve (4), 1st notch.
19	192	27.8	7.16	" "
20	192	13.9	3.57	Level, "
21	210	16.2	4.56	" "
22	192	15.4	3.96	" "
23	229	0.	....	Stop, power off.
24	208	16.2	4.57	Level, 1st notch.
25	208	50.9	14.19	Starting, "
26	184	34.7	17.43	Curve (5), 1st notch.
27	208	83.3	*23.22	Start, slight down grade, 1st notch.
28	210	25.5	7.17	Level, dirt on track, 1st notch.
29	192	16.2	4.17	" " " "
30	188	97.2	*24.49	Start, 2d notch.
31	204	20.8	5.68	Level, "
32	196	27.8	7.30	Curve, down grade, 2 1 notch.
33	200	25.5	6.90	" " "
34	200	27.8	7.45	Up grade, 2d notch.
35	212	39.4	11.19	Curve, down grade, 2d notch.
36	192	97.2	*24.95	Start on curve (6), 1st notch.
37	208	26.8	7.47	Up grade, 1st notch.

Striking out 15, 27, 30 and 36, the average horse-power used was 5.96.

In running at average speed on a level track, from 3 to 4 horse-power was used.

Efficiency Tests.—The efficiency of any motor is the ratio of the useful work obtained to the power expended in obtaining it; thus in the case in point.

$$\text{Efficiency } (E) = \frac{\text{Electric horse-power taken.}}{\text{Mechanical horse-power given out.}}$$

To obtain the efficiency of the motors, the electric horse-power taken was calculated in the same manner as in the car tests; but to get at the mechanical horse-power given out at the wheels, a differential rope Prony brake was used. The motor tested was a new one that had not been placed under its car; the rear axle was disconnected from the frame, and the frame, together with the motor, gearing, driving wheels, etc., was blocked up off the ground on dry pine blocking and braced in different directions to hold it steady when run. The electric connections were made in the same general way as in the car test, a special switch and connections being used.

The Prony brake was applied to the driving wheel nearest the gearing, to neutralize the bending strain on the axle and to get rid of the necessity of using two brakes. The construction of the brake was as follows: a secondary flange consisting of an iron ring of the exact size of the tread of the driving wheel was forced on its outer edge, making a groove about 2 inches wide and half an inch deep around its circumference; a half-inch rope was given two turns around the circumference of the wheel in this groove, and its ends brought up vertically and attached to two spring balances, which were suspended from a framework directly over the tread of the wheel. These spring balances were attached to long threaded hooks with thumb screws on the upper side of the framework by which the tension on the rope could be varied at will.

In making the tests, tension was put on the brake by means of the thumb screws; and when the revolutions had become constant, the test was started. Simultaneous readings of the brake, on and off sides, voltmeter and ammeter, were taken at regular intervals, and the total number of revolutions was observed at the finish.

The following table (Table No. 3) gives the results of the tests. The first column gives the number of the test; the second, the duration of the same; the third, the total revolutions during the test; the fourth, the average tension on the main brake rope; the fifth, the average tension on off side of the brake rope; the sixth, the mean pull, being the difference between column 4 and column 5; the seventh, the average voltage; the eighth, the average current in ampères; the ninth, the electric horse-power taken, being $\dfrac{\text{current} \times \text{electromotive force}}{746}$;

the tenth, the mechanical horse-power given out at the circumference of

the wheels, as expressed by the formula, $\dfrac{2\pi r n P}{33\,000}$, in which

r = radius of driving wheel measured to center of rope.

n = number of revolutions per minute.

P = mean pull.

the eleventh, the efficiency $= E = \dfrac{\text{electric horse-power}}{\text{mechanical horse-power}}$; and the

twelfth, the description of the test.

TABLE No. 3.

No.	Time, Min.	Rev's.	Brake pull, pounds.			Volts, average.	Ampères, average.	H. P. electric.	H. P. mech.	Efficiency.	Remarks.
			On.	Off.	Mean.						
1	2	168	49	0	49	189	16.2	4.10	1.01	.24	1st test; at 1st notch
2	2	165	89	2	87	196	17.4	4.28	1.77	.41	2d " " " "
3	1	75	129	10	119	187	19.2	4.81	2.19	.46	3d " " " "
4	2	160	129	15	114	213	21.5	6.41	2.24	.35	4th " " " "
5	2	199	109	7	102	192	17.1	4.39	2.49	.56	1st " " 2d "
6	2	199	109	17	92	198	19.4	5.15	2.25	.44	2d " " " "
7	2	189	149	22	127	183	21.7	5.32	2.95	.56	3d " " " "
8	2	211	134	24	110	177	31.6	7.49	2.85	.38	1st " " 3d "
9	1	113	134	24	110	199	33.3	8.88	3.06	.34	2d " " " "
10	2	233	169	40	129	194	32.8	8.53	3.70	.43	3d " " " "
11	1	115	189	38	151	202	32.	8.67	4.27	.49	4th " " " "

It can be seen from the table that the best results were obtained when running on the second notch, and at about average speed and power, as shown by the car tests, which fact bears out the statement of the builders that the motors were designed to be run in that way.

MOTIVE POWER FOR STREET RAILWAYS.*

By ALFRED F. SEARS, M. Am. Soc. C. E.

READ SEPTEMBER 7TH, 1892.

It is a part of the experience of every civil engineer who has had to deal with the problem of traction for urban lines, that, as yet, no satisfactory solution has been presented. In every system thus far popularized by use on street railways, there is a vast waste of power in transmission or transformation, or in hauling cumbersome loads of fuel and water.

It has lately become the duty of the writer, in the interest of a street railroad company, to investigate with patient care the subject of motive power for such works. The problem has been: What is the most economical motive power for street railroad traffic; and what is the cheapest application of that power, compatible with the conditions

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* Discussions on this paper will be received until November 15th, 1892, and published at a later date.

demanded by the preferences and prejudices of the public in cities; whether as residents, subject to its presence, or as habitual patrons, using the system as a means of locomotion?

In studying the matter, he has been impressed with the belief that a collation of the data used, together with the conclusions reached, in as brief form as consists with its clear elucidation, will be of value to the profession; also, that a discussion of these conclusions by members of the Society will serve to either confirm the deduction or reveal a more correct solution, so that in any event the general good will be advanced by offering this paper to their attention.

There should be no difficulty in recognizing a primary fact, viz., that no system of motor, suitable to all circumstances, has yet been devised and probably never will be. It is a generally accepted doctrine that steam, and its derivatives, electricity and compressed air, are invariably cheaper than animal power. It is, indeed, difficult to imagine a situation where the opposite holds true. And yet, in the Republic of Mexico, a railroad, highly finished in every detail of road-bed, masonry and superstructure, carries mail, passengers and baggage from Esperanza, on the Mexico and Vera Cruz line, to Tehuacan, a distance of 50 miles, in five hours with animal power only. Freighting is, of course, done at a slower pace by the same kind of power. Nor does any other motor seem practicable. There is neither coal, petroleum nor wood in all that region; while freight charges on the line from Vera Cruz, by which coal might be imported, prohibits the transportation of fuel. On the contrary, provender is abundant and mules are cheap. A similar condition obtains in the City of Mexico and its suburbs, where there is no native fuel, and all urban and suburban lines employ animal power exclusively; and so far as can now be seen, this must remain the condition.

But generally the humanity of man has joined hands with his cupidity, and the civilized world seems bent on finding a substitute for animal power to do its work of transportation. The conclusion reached after a careful, and, it is believed, unbiased study is: (1) That steam without transformation is the cheapest power that can be used where fuel can be had for reasonable cost, and is destined to be the street motor of the future. (2) That electricity is the cheapest where it can be produced by water power; and in such cases will remain, as now, a popular motor with passengers and the public. (3) That compressed air

engines will be the cheapest available method and the best, for dealing with all the conditions of tunnel and underground lines, where electricity is not practicable.   (4) That cable roads are best for the steep street sites of hilly towns, and by saving the immense cost of grading, are the most economical, if not the only practicable, for controlling the traffic in such situations; but that they are expensive, inconvenient and dangerous on level streets in business thoroughfares.   These are the conclusions arrived at in seeking a power that shall produce the largest dividend and the quickest return.

Mr. Henry C. Adams, the distinguished statistician of the Interstate Commerce Commission, has inspired and supervised a comparative statement, made directly by Mr. Charles H. Cooley of the United States Census Bureau, and published in " Census Bulletin No. 55," touching the relative economy of cable, electric and animal motive power for street railroads.   This document is preliminary to another, which is to contain complete statistics of street railroads, but is not yet ready.   Bulletin No. 55 embraces statistics of fifty lines of street railroads, ten of which are operated by electricity, ten by cable and thirty by animal power.   We are not told the names of these lines, nor are we given any hint betraying their locality, or the price of fuel or provender.

On only one line are all three classes of power availed of, and so uniform are the resulting costs of operating per car mile and per passenger carried, that we seem to have met a case of peculiar wisdom in selection, though probably the animal power remaining is a necessity of the problem, to be hereafter improved.

Thus the tabulated results are as follows :

Power Used.	Cost per Car per Mile.	Cost per Passenger Carried.
Cable	9.39 cents.	3.48 cents.
Electric	9.77 "	3.75 "
Animal	9.21 "	3.49 "

While the operating expenses appear at first view so nearly equal, a closer inspection betrays but little difference in the work done; for the cable line in 11.69 miles of street contains but 566 feet of 14 per cent. grade, against 475 feet of 5.2 per cent. grade on 4 miles of the electric line.

If we take the average of the fifty roads under consideration, we find no great difference in the average cost of carrying a passenger by any of these three motors over their different lines.   Thus the opera-ing expense is :

Per passenger carried by cable railway........3.22 cents.
"   "   "   electric "   ....3.82   "
"   "   "   animal "   ....3.67   "

Being only a trifle different from the corresponding items on the line, which in its own single limits includes the three classes of power.

A little farther examination demonstrates that the average aggregate cost of carrying a passenger over all three sections of the composite line, is almost exactly the same as a similar duty performed on three average sections of the entire cluster of fifty lines; being 10.71 cents in the former case, and 10.72 cents in this; an absurdity illustrating the fallacy to which the engineer is exposed in the use of averages, and also the comparative worthlessness of the passenger unit in calcula-tions of this class for the United States, where the distribution of population is a factor so constantly changing in amount and direction, that no prediction nor estimate of it can be made with fair degree of certainty.

The car mile unit seems to be the correct representative of work done on a line.   The car must be moved whether full or empty, and the difference of duty on electric and animal roads, between a full car and a car but half full is not appreciable in the cost of operating, since, in both cases, the driver will make up expenditure of power in speed when his lighter load will permit.   This is not practicable with the cable lines, where the velocity of the car is uniform and is that of the cable.   When steam, however, is directly applied, as in locomo-tives and dummies, the expenditure may be more exactly proportioned to the load.   This advantage will also accompany the compressed air motor, and perhaps the storage battery of the electric systems.

Examining the car mile cost of operating the fifty lines under con-sideration, we have :

Kind of Power.	Length of Line.	Number of Cars.	Cost per Car Mile Run.
Cable,...................	143 miles.	583	14.12
Electric............	67 "	78	13.21
Animal...................	550 "	1 500	18.16

These figures, again, are averages, and in the case of the ten cable
roads, cover limits extending from 9.4 cents to 22 cents; in the ten
electric lines, the figures range from 8.34 to 23 cents; and in the animal
lines from 9.1 to 27 cents; another illustration of the fallacy of averages,
which may be made still more apparent from the figures of these tables;
for from the thirty animal lines it is possible to select ten, of which the
average cost per car mile is only 10.82 cents between limits varying
only from 9.1 to 13.89 cents.

An important difference in the operating expenses, appears in the
item, "Repairs of Road-bed and Track," and for obvious reasons. Thus
it cost $709 per mile of track to keep the cable roads in shape a year;
while the same item for the electric lines amounts to $190, and for the
animal lines $430. But in this connection we must bear in mind that
the animal lines moved 1 500 cars, to only 78 and 580 moved by the
electric and cable lines respectively.

From figures obtained by the Census Bureau and published in the
Street Railway Review of January, 1892, it appears that, up to the date
of compilation, the operating expenses of lines in the United States
had been as follows :

Kind of Power.	Operating Expenses per cent. of Earnings.	Dividends on Stock, per cent.	Surplus Capital on Hand, per cent. of Capital Stock.
Cable....................	67$\frac{7}{10}$	10.15	11$\frac{7}{10}$
Electric	70$\frac{1}{10}$	5.59	2$\frac{1}{2}$
Animal	73$\frac{7}{10}$	7.03	10$\frac{1}{2}$
Steam....................	57$\frac{7}{10}$	6.03	5$\frac{7}{10}$

It is proper to observe at this point, that cable roads are built only
where the heaviest traffic is already assured, while the steam motor in
its various forms has been confined thus far to the most sparsely
peopled regions demanding street railway accommodation, and is
almost limited to Southern and Western States, where are found 351
steam motors out of a total of 391 in the United States.

Formulating the results given by the "Census Bulletin" No. 55,
and calling the cost of animal power 100, the cost of cable will be 77.74,
and of electricity 72.74.

In 1889, Captain Eugene Griffin, U. S. Engineers, arrived at a
different result, for, calling the maintenance of animal power 100, he
demonstrated the cost of cable traction to be 79.5, and of electricity

46.5.   At about the same time, Mr. F. H. Whipple, in a book on electric railways, determined that if the cost of animal power be designated at 100, then cable traction would stand at 118, and electricity (overhead wire) at 83.   It would appear that these figures were reached by accepting as accomplished facts the promises of the electrical enthusiasts, who, and it is said feelingly, had already acquired the noble art of making hay while the sun shone.

Suppose we formulate in the same way the latest data giving the proportion of operating expenses to the earnings of all or nearly all the roads in the United States, as taken from the *Street Railway Review*.   We shall then have the proportional cost of power as follows :

Animal lines which spent $73\frac{7}{10}$ per cent. of their earnings, say 100

Electric lines which spent $70\frac{4}{10}$ per cent. of their earnings, say .... $99\frac{5}{10}$

Cable lines which spent $65\frac{7}{10}$ per cent. of their earnings, say .... $89\frac{1}{10}$

Steam lines which spent $57\frac{3}{10}$ per cent. of their earnings, say .... $77\frac{8}{10}$

There seems but little reason to doubt, that if steam could be inoffensively applied, it is the proper substitute for animal power on the streets of cities.

In the cable road there is a vast expenditure of power exhausted before any useful load is moved.   In the ten cable roads of the bulletin, the average length of track, which is also the length of the cable, is more than 14 miles ; and the power must drag a load that weighs from 60 to 80 tons, to be moved 6 miles per hour, before any paying duty is performed.   The work done to keep this mass in motion may be imagined when we observe the cost of repairing cables and pulleys, which for these ten lines of 143 miles amounted to $283 338 in a single year, or $1 981 per mile in that time.

The electric system wastes in losses a very important percentage of the steam generating power.   How much, it is not possible to say, without raising the hue and cry of contradiction.   Personal observation leads the writer to believe that the traffic which feels 50 per cent. of the steam power generated at the central station, has extraordinary luck in its successful economy.   With the steam locomotive, whether

disguised in the dummy or otherwise, there is loss in carrying the load of fuel and cold water, as also in the vehicle or magazine which contains them.

After careful investigation among a tiresome mass of inventions, some of which show much ingenuity, there have been found two systems which seem to promise, in one form or the other, and perhaps in both, the street motive power of the future. They are, in one case, engines moved by compressed air, and in the other by compressed steam, or water of such a temperature and under such pressure that, when released, it becomes steam, ready for work. The motors for urban use may in both cases be called small packages of condensed power. An advantage of both engines will consist in the fact that they can be built to do a fixed maximum of duty of defined limit, and that this limit is so restricted as to prohibit an attempt to accomplish too much. Thus, the compressed air engine, once charged, is good for a distance of 10 miles without recharging. The compressed steam motor will go 40 miles without reinforcement.

A Toledo street railway company, after experimenting with the compressed air motor, is so far satisfied with the results obtained that a complete plant is being installed in that city. Prof. D. S. Jacobus, of Hoboken, saw the system in operation at Nantes and Vincennes, France, where the roads are 5 and 7 miles long respectively, and have been successfully operating this system for twelve years. At the New York meeting of the American Institute of Mining Engineers held in September, 1890, that gentleman read a valuable paper, which has been published by the Institute and is the authority for what is here said of the system.

As that paper is accessible to the members of this Society, is sufficiently illustrated, and is carefully elaborated in details, only the general facts are here given, touching its construction; leaving it for those personally interested in the subject to study the minor features in the paper mentioned, or on the ground, at Toledo. In the motor car, two small engines are connected so as to rotate the front axle of the car, a reversing lever being used to alter the cut-off and change the direction. The compressed air is held in tanks under the bottom of the car and is admitted to the engine cylinders after passing through a mass of hot water, which leaves the charging station at the temperature of about 300 degrees Fahr., and is reduced to 212 degrees when it has

returned to that point.   The engine cylinders are 5½ x 10½ inches and the compressed air is charged in its retorts at about 425 pounds per square inch.

Prof. Jacobus estimates the cost of compressed air motive power, as compared with horse traction in Nantes, where there is experience, and great economy of air is observed in handling the engine, to be such that if the cost of animal power is put at 100, the cost of the compressed air power will be 63.33.   He is of opinion that for a time this power must be limited to localities unencumbered with snow; and believes that for underground, mining and tunnel service its ventilating capacity will make it of great practical value.   This system is not only in operation at Nantes and Vincennes, it is also in use at Nagent, near Paris, where each motor draws after it a train of two trailers; at Marseilles, where the cars are operated under a storage pressure of 1 200 pounds per square inch ; and at Berne, Switzerland.

An interesting and fairly conservative account of the performance of the compressed air engine at Toledo, may be found in the *Chicago Street Railway Review* for January of this year.   Snow and cold weather seem not to have troubled the experiment, and the old difficulty of frozen air valves no longer exists.   The manufacturers claim a trip of 9 miles length as the limit of duration of power, and there seems but little chance of increasing this capacity without lengthening the car or raising its floor higher from the street level.

The last mode of applying power to which attention is invited, is in the form of compressed steam and was studied twelve years ago. Later, the invention was laid aside for want of means to improve the original excellent idea; and during the past year, the tendency to recur to some mode of applying steam directly, in the motor, has led to such improvement on the original design that a fair ground of hope exists, that we have, at last, the motor best adapted to urban travel, in what is still known as the Angamar motor, now the subject of experiment in Chicago.   Several other plans are also now being proved in that city, viz., the Patton gasoline engine, wherein a portable dynamo is operated by steam and the product transferred to an electric motor connected by the usual method with the wheels; the Judson compressed air engine, having three conduit charging stations for each 5 miles of road; the Belgian motor, which Mr. Yerkes bought in Belgium and has been trying upon one of the North Side roads, a rather small and some-

what complicated locomotive, referred to by the Baldwin Locomotive Works, in their letter following, as being "too small"; the Connolly gas motor, which has inadequate power unless unreasonably large for street use; the Prouty motor, a small steam engine, with inadequate power. These and indeed several other devices serve to illustrate the variety of invention all tending in one direction.

When the writer's attention had been called to the Angamar motor, he wrote to the Baldwin Locomotive Works, to learn the opinion of men, who have probably built more street dummies and very small locomotives than any other factory in the world, to learn their opinion of a machine with such pretensions. They replied:

"We believe that such a motor as you describe can be constructed, and that, if satisfactorily developed, a large demand will result. Many roads which are at present operated by electricity, as well as other roads which are unable to obtain electrical franchises and cannot make the expenditure involved by the cable, will be likely to adopt them."

Those gentlemen also said in the same letter: "The demand for a steam motor is so strong, that, notwithstanding the admitted objections to these machines, we have constructed upward of three hundred of them.

"During the past winter our attention was strongly drawn to the desirability of designing a condensing motor. We built an 18-ton compound motor in which we sought to accomplish the following: (1) To utilize the steam by expansion to so low a tension that its escape from the cylinders would be accompanied by little or no noise, and at the same time such expansion would so considerably reduce the temperature of the escaping steam as to render it easier to condense; (2) to provide a condenser large enough to condense all the escaping steam ; (3) to rely entirely upon natural draft, excepting when unusual power was required, in which case the steam could be diverted from the condenser and discharged in the ordinary manner through the exhaust nozzles into the stack.

"This motor was measurably successful and accomplished the results intended, but we did not succeed in entirely avoiding the show of steam in bad weather. This was probably due in part to the large size of the motor, requiring the generation of so considerable a volume of steam as to render its condensation more difficult. We look for more satisfactory results from a similar experiment with a smaller motor. Meanwhile that motor was purchased for noiseless switching service in Wilmington, N. C., where it is doing satisfactory work.

"Some time since, the North Chicago Street Railway Company imported a Belgian motor, which is said to accomplish all the results which we sought. We have contracted to duplicate it, and, of course, guarantee equally satisfactory results. This motor is, however, too small. We have also agreed to build, from our own design, a somewhat more powerful motor with which we have guaranteed equally satisfactory results."

On the strength of various representations, a series of experiments have been made at the writer's request, with a motor built by the

Kinetic Power Company in Chicago, having the Angamar boiler, the results of which are here presented; they were obtained from the observations of both interested and disinterested parties, and the writer places such confidence in them that he has recommended the system to his company as the proper and only solution of the street-motor problem in all ordinary cases.

The Angamar motor car, in size and appearance, resembles the grip car of a cable road. As above said, this motor is a contrivance for using compressed steam. Water is heated at a "charging station," so called, to the temperature of 387 degrees Fahr. (200 pounds steam pressure). This station consists simply of furnace and boiler—preferably the upright tubular—which should be of comparatively large capacity.* A plant of 400 horse-power will be ample for about one hundred motors of 50 horse-power each. This stationary boiler is tapped on the low water line for connection with the retort of the motor, and also in the dome. Water may thus be charged above, or steam, or the two together when it is requisite to quickly produce the maximum pressure in the retort. The latter, with all its connections of pipes, dome and firebox, is thoroughly jacketed to prevent loss of heat by external radiation. When the retort of the motor has been charged with hot water and steam, a few shovelfuls of burning anthracite coal are thrown into the firebox.

The motor now under experiment in Chicago has a pair of 9 x 10 cylinders; the retort, having a capacity of 263 gallons, is charged with 160 to 170 gallons of water, heated, as already said, to nearly 400 degrees Fahr., and is rated by indicator test at 43 horse-power.

By this system it is seen in experience that while the quantity of water in the retort is evaporated and the rapidity of steam-making tends to increase, the fuel in the firebox decreases by consumption in amount and heating power, and thus reduces the tendency to excessive pressure. As the highly heated water is conveyed from the charging boiler to the motor, it first becomes steam vapor and as such enters the retort; but as the injection is continued, a water level becomes established, showing that a portion of the steam under such pressure has returned to the form of water.

Accounts of trips with this machine now in Chicago have been received in which it is reported to have made 20 miles with one charging on the track of the West Chicago Street Railway Company.

The retort for holding water contains 263 gallons, of which about 160 are, at the beginning of the trip, for evaporation. The driving wheels are of 31 inches diameter, and thus 1 303 cylinder volumes of steam per mile are used up by each of the two cylinders. Or, the capacity of the cylinders being 636 cubic inches, the volume of steam used per mile will be $636 \times 2 \times 1\,303 = 1\,657\,416$ cubic inches.

The 160 gallons of compressed hot water contain an energy of $37\,000$ cubic inches $\times 1\,642$, the capacity for expansion, or $60\,754\,000$ cubic inches of steam. If this value were maintained, the motor has a traveling ability of about 37 miles under the charging above mentioned. The actual experience in a course of eight days shows that it may be relied on, starting with 155 pounds pressure, to make 20 miles without new charging, and return to the shop with a pressure of 142 pounds after such a run; using 30 pounds of anthracite coal in the trip and carrying eighty passengers on motor and trailer, moving sometimes at 18 miles per hour.

The machine appears easy to manage and does not require any more intelligence in the driver than the ordinary electric car. It shows no smoke, and where anthracite coal is not to be had, it is believed that the simplest arrangement of kerosene burners will maintain the condition of heat required. An improved arrangement has been designed by which condensers are to be added to prevent the slight noise that is now heard in the escaping steam. The weight of a motor car, which, as before stated, is arranged much like the ordinary cable grip car, and like it will haul one or more trailers, is about that of the ordinary electric motor's weight, from 5 to 8 tons.

Captain Charles F. Thomas, the Constructing Engineer of the Kinetic Power Company, estimates the cost of power alone per car mile at $\frac{55}{100}$ of one cent, which is much less than one-third the cost of electric power on the West End system of Boston.

THE PROTECTION FROM CORROSION, OF IRON-WORK USED AS COVERING FOR RAILROAD TUNNELS.

By JAMES G. DAGRON, M. Am. Soc. C. E.

READ JUNE 10TH, 1892.

WITH DISCUSSION.

The main connection between the Baltimore and Ohio, and the Philadelphia and Reading Railroads, through the City of Philadelphia, is effected by means of the Schuylkill River East Side Railroad, whose tracks skirt the east bank of the Schuylkill River from below Gray's Ferry to Callowhill Street, and there, diverging from the river front, pass through a tunnel under 25th Street, at the base of the Fairmount Reservoir, and an open cut along Fairmount Park to Park Junction, near Girard Avenue, where the connection is made.

The headroom under the street at the ends of the tunnel, not being sufficient to permit the use of an arch, the street was supported by a buckle plate flooring carried by transverse wrought-iron girders resting on the side walls of the covered way. The total length of the tunnel is 2 308 feet, 1 583 feet being arched, and 482 feet at the south end and 243 feet at the north end being covered as described above. The construction of this part of the tunnel is clearly shown on Figs. 1 and 2 and Plate LIII, in the last volume of the *Transactions*.
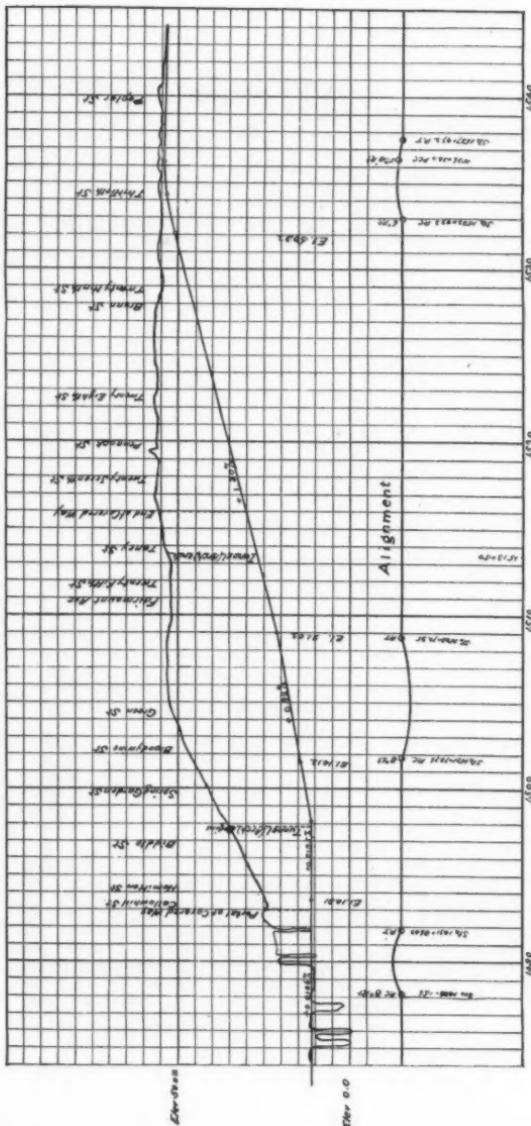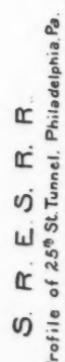
FIG. 1.
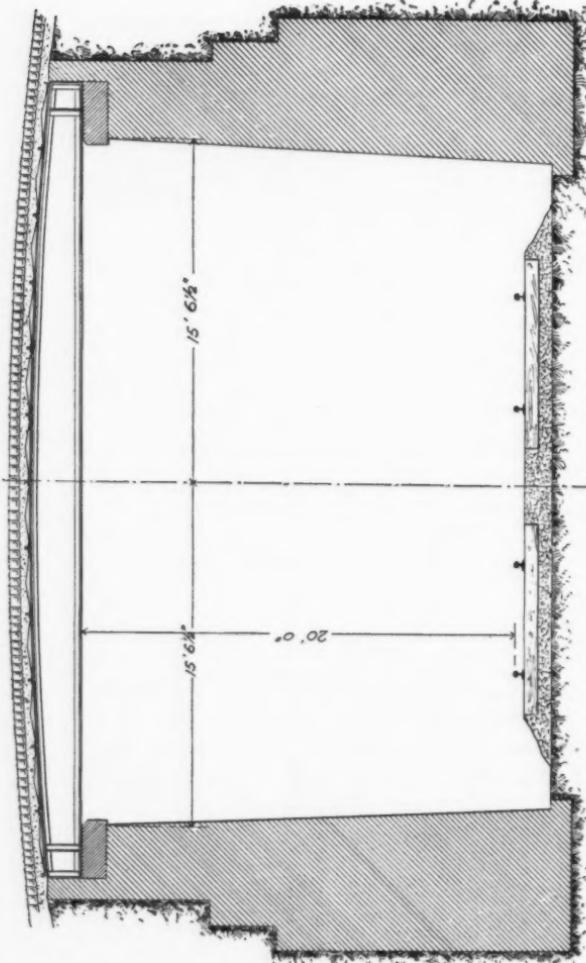
CROSS SECTION OF 25TH STREET COVERED WAY   B. & O.R.R.   PHILADELPHIA PA.

Fig. 2.

SCALE.
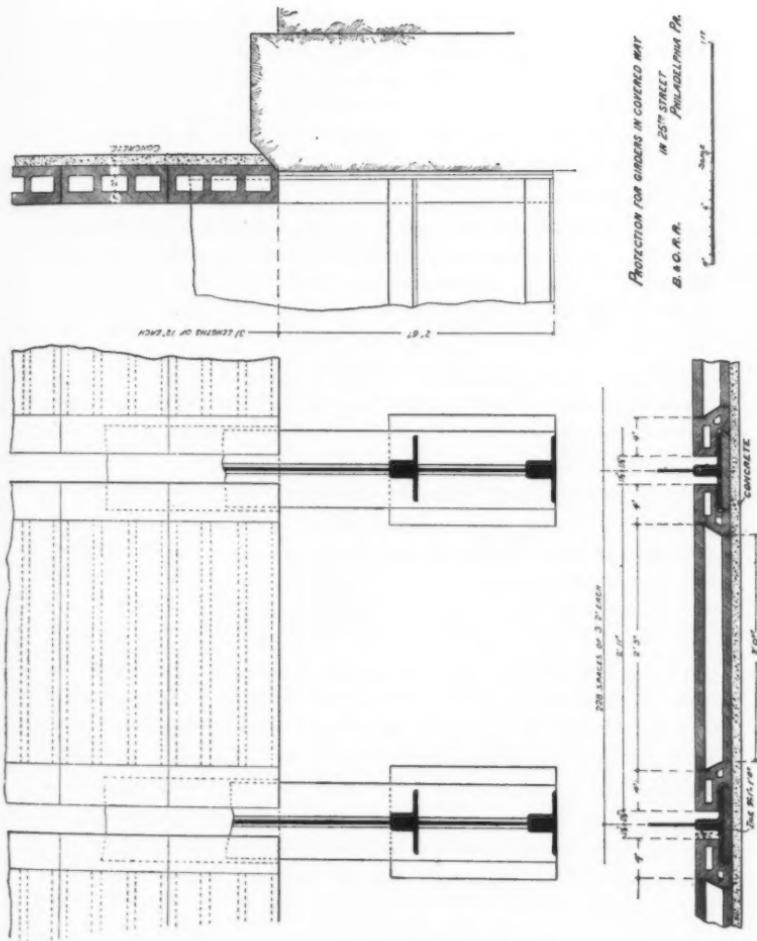
FIG. 3.

The running of trains through the tunnel began in September, 1886. During the winter of 1890–91 the attention of the author was called to the very serious corrosion of the girders which was taking place under the combined action of steam and gases from the loco-motives passing through the tunnel, and he was instructed to devise some means by which this action could be avoided, as its further con-tinuance, in time, would have endangered the safety of the work.

It is the purpose of this paper to present a concise account of the method adopted. This plan consisted essentially in hermetically seal-ing the iron-work from the access of steam and locomotive gases by a flat arch of hollow firebrick tiles, extending from flange to flange of the girders, and running their entire length on to the coping of the side walls, a thick coating of cement mortar being laid on the under-side of the tiles, the details being shown on Fig. 3.

The work was carried on as follows: the iron-work was thoroughly scraped to remove every particle of rust, and the girders given two coats of asphalt paint, the skewback tiles were then placed in posi-tion on two contiguous girders, and the keystone tiles let into place. When a certain number of lengths of tile had been put in, a workman, whose duty it was to thoroughly seal up each joint with cement mortar, was sent in through the space enclosed by the two girders, buckle-plate covering and flat arch; the last keystone tile was left out for his exit, and was in turn, after having its edges cemented, dropped into position. After sealing the joints from the outside, a coating of cement mortar about 1 inch thick was laid on the under-side of the tiles, corrugations having been left in them to ensure a good adherence of the mortar. The work was necessarily tedious and slow, as it had to be carried on from a raised platform over the tracks, with trains rushing continually through the tunnel, the men being obliged to work on their hands and knees, and in the dim light afforded by oil lamps. The work was carried on under the supervision of Mr. B. Frank Richardson, M. Am. Soc. C. E., and there is every reason to believe it will accomplish the purpose intended.

The cost of the work, including the painting of the iron-work, was $1 40 per linear foot of flat arch, or $0 44 per square foot of flat arch; or, if the cost of painting the iron-work is omitted, $1 13 per linear foot of flat arch, or $0.357 per square foot of flat arch, which is a reasonable figure considering the difficulties under which work had to be done.

DISCUSSION.

DESMOND FITZGERALD, M. Am. Soc. C. E.—Has Mr. Dagron any direct evidence that cement mortar has preserved the iron ?

JAMES G. DAGRON, M. Am. Soc. C. E.—The use of the cement mortar is not to preserve the iron. The iron was coated with two coats of asphalt paint; the object of the construction adopted was to prevent the gas and steam from the locomotive coming in contact with it. The idea I meant to convey was this, that after these tiles were laid, a workman was sent in to seal the joints; in other words, fill the joints with cement. The space above the tiles is not filled with anything, except that their upper surface is flushed with cement.

We have noticed, since this work has been put in, that the tunnel is much freer from smoke than it formerly was. The reason for this may be seen by looking at the profile of the tunnel, Fig. 1. The inside of the flanges is protected by the tiles.

The buckle plates were galvanized; and when we went in to examine them, they were coated with soot, which, being removed, showed the buckle plates to be entirely free from any action of the steam or gas. The girders were seriously attacked, the rust being apparently one-eighth of an inch thick.

CHARLES MACDONALD, M. Am. Soc. C. E.—The necessity for some means of protecting iron-work, where it is exposed to gases from locomotives, has long been felt; galvanized iron sheathing soon disappears; buckle plates prolong the situation, but yield rapidly, even under the best of care ; and, in some cases, notably with the overhead street bridges in Boston, the advisability of sheathing with wood was seriously considered.

The method described in the paper seems to be very simple and effective. The wonder is, that so much has been accomplished at such a moderate expenditure, considering the extremely complicated conditions. In a new construction where tiles could be placed without the restrictions involved in the present case, the cost could, doubtless, be reduced to a figure which would commend it to a general practice.

Mr. DAGRON.—I think this method will be cheaper, although I am not prepared to give figures at this moment.

Mr. FITZGERALD.—If there is any chance of cracking, I don't see how you make sure that the gases are not eating the iron out.

Mr. DAGRON.—We are relying on this ceiling which is continuous. All the joints were thoroughly sealed, and the work was very carefully looked after. A coating of 1 inch of cement mortar was put on below the tiles, over the whole surface.

A MEMBER.—I think the idea is an excellent one. Some remarks have been made looking to the use of galvanized iron. I don't believe

it is practicable to use galvanized iron in such a case.   We have abandoned its use on our railroad in any such form.   We had been in the habit of manufacturing cornices of galvanized iron ; they have been totally destroyed within two years, and we have substitued wooden cornices.

C. B. BRUSH, M. Am. Soc. C. E.—Does not the cement or concrete 1 inch thick crack ?

Mr. DAGRON.—It does not.   I was in that tunnel over three weeks ago, when the work had been finished about four months, and there is no evidence of any cracking.   We examined the ceiling very carefully ; it seemed to adhere to the bottom of the tiles very well.

Mr. FITZGERALD.—The plastering is not exposed to the same variations of temperature that it would be outside of the tunnel, where a large surface would be very apt to crack owing to such changes.

Mr. DAGRON.—It would be very apt to, but the temperature of the tunnel is much more uniform than that of the air outside, and there has been no such difficulty.

I think that in the case of a bridge, such as mentioned by Mr. MacDonald, instead of using a coating of cement underneath, I would use a coat of asphalt ; it would act in the same way.

Mr. FITZGERALD.—The cement mortar, I suppose, really does preserve the iron from the effects of rusting.   Not long ago, I heard Mr. Robert Moore describe a method of treating bolts for bridges, by which, instead of using lead or sulphur, Portland cement mortar was used, and it was found that the bolts were much better preserved in every way against rust and against pulling.

J. P. FRIZELL, M. Am. Soc. C. E.—Why would not the cement applied directly to the iron-work protect it from this corrosive action ?

Mr. DAGRON.—I should think it would be harder to get the cement to adhere to the iron-work.

Mr. FRIZELL.—As to the adhesion of cement to iron, it is well known that iron pipes coated with cement are laid down by the mile.   These have been taken up in many places, but not because of lack of adhesion of the cement.   I have taken up pipes that had been laid many years, knocked off the cement, and found the iron as bright as new.   It occurs to me, therefore, that in the case treated of in this paper, it would be well worth while to consider the propriety of applying the cement directly to the iron.   The cement certainly preserves the iron from rust.

There is one question I want to ask : Whether the corrosive action of the gases that escape from the locomotive is not largely due to the heat; whether the hot gases would not affect iron much more strongly than if they were cool ?   Do your observations confirm that supposition ?

Mr. DAGRON.—I should think that the action would be much greater at a higher temperature than at a lower temperature.

Mr. FITZGERALD.—I think the gases have a great deal to do with it.

THE STRENGTH AND WEATHERING QUALITIES OF ROOFING SLATES.

By MANSFIELD MERRIMAN, M. Am. Soc. C. E.

READ SEPTEMBER 21ST, 1892.

WITH DISCUSSION.*

The slate formation of Northampton County, Pennsylvania, lies along the southern side of the Blue, or Kittatinny, Mountain, having an average width, measured on the surface in a northwest or southeast direction, of 7 or 8 miles. The section given in Fig. 1 shows the geological position of these deposits. Beginning at the South Mountain is seen the Archæan gneiss, the oldest of the sedimentary rocks. Above this is the Silurian limestone, which underlies in turn the slate belts. These belong to the formation called by geologists the Hudson River slates, and here they dip toward the northwest and have a thickness, perpendicular to the dip, of about 6000 feet. Above the slate is seen the Oneida sandstone of the Blue Mountain.

The South Mountain has an elevation of about 750 feet above tide,

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* Additional discussion on this paper if received by November 15th, 1892, will be published in a subsequent number.

while that of the Blue Mountain is 1400 feet. Between them is included the Kittatinny, or Great, Valley, which extends from New Jersey to Georgia with essentially the same geological formations, and which is noted for its fertile soil, its iron ores and its slate quarries.
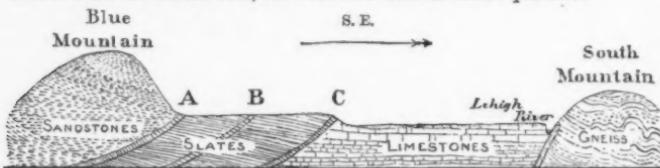
FIG. 1.

The geological structure of the slate formation is very complicated, and in a particular quarry neither the original beds of stratification nor the planes of cleavage may be at all related to the general dip. This is due to the upheavals, overturnings and pressures to which the slates have been subjected since the period when they were deposited as horizontal layers of clay in quiet waters.

The original planes of stratification are usually seen in any quarry definitely marked by the seams or ribbons which traverse the rock in directions approximately parallel, and which differ in color and composition from that of the slate proper. The thickness of the ribbons may be an inch or two, although some are much thinner, and their distance apart varies from a few inches up to 6 or 8 feet. Between the ribbons are the beds of original stratification, each bed being usually of uniform quality and composition, but often differing in quality from adjacent beds.

The planes of cleavage of the slate are not parallel to the ribbons, but make different angles with them in different quarries. The direction of these planes is supposed to be perpendicular to that of the pressures which consolidated the slate formation in its present position. In some quarries the cleavage planes are almost horizontal, but usually they dip toward the southeast with inclinations varying from 5 to 50 degrees.

In Fig. 2 is shown a typical view of a vertical section of the top of the slate rock where the ribbons are seen curved and the cleavage planes are parallel. In any particular quarry the cleavage is usually uniform in direction, while the ribbons are often curved and contorted and sometimes the beds are folded over upon each other.

The slate deposits near the top of the formation outcrop at *A* in Fig. 1, near the base of the Blue Mountain; while those near the bottom of the formation outcrop at *C*, several miles away from it. The upper beds, comprising those from *A* to *B*, have a total thickness of about 1 500 feet, while the lower beds from *B* to *C* are probably 4 000 feet thick or more. The upper beds produce the soft slates which are used for roofing and for school slates, while the lower beds produce the hard slates which are used largely for sidewalks and steps. In the upper beds the ribbons are soft and of inferior quality to the slate proper, so that they are generally cut out in preparing the stock for market. In the lower beds the ribbons are often harder than the slate itself and need not be excluded. The hardness of the slate is usually greater in the lower beds, these having been subjected to a greater pressure than the upper ones.
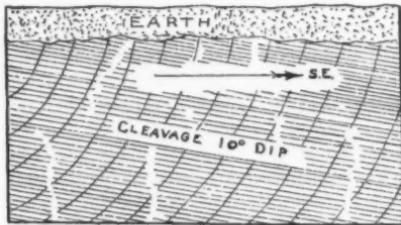
FIG. 2.

The first slate quarry in Pennsylvania was that of J. W. Williams, in Upper Mount Bethel Township, Northampton County, which was opened in 1812. The Report of the Second Geological Survey, published in 1883, mentions and partially describes nearly one hundred quarries in this country; but it should be said that many of these are merely holes in the ground, half filled with water. Quarries are worked at Slateford and Portland; at Bangor, East Bangor and West Bangor; at Pen Argyle; at Chapman's, Danielsville and other localities, which produce large quantities of slate for sidewalks, roofs, blackboards, mantels and other purposes.

The slate deposits are covered with earth and gravel to the depth of from 10 to 50 feet. The first operation in opening a quarry is to strip off this surface-material and the weathered outcrop of the rock. The slate is then quarried by drilling holes at right angles to the

cleavage planes and blasting out large blocks. The holes are made in such positions and the blasts so adjusted in intensity, that the blocks are only moved a few feet horizontally and are not shattered. These are broken into smaller blocks, say, about a foot or two thick, 2 or 3 feet wide and 8 or 10 feet long, which are hoisted out of the quarry by the derrick and then run on small cars to the shanties where they are to be dressed. The quarrymen are divided into gangs of from six to twelve men, each gang having a contract to produce a certain amount of finished slate at a certain price. The men provide their own tools and powder, apportion the different parts of the work among themselves according to their different capacities, and deliver the finished slate in the yard, the company having only to hoist the blocks out of the quarry, keep it free from water, remove all debris, exercise a general superintendance, and inspect the completed slates.

Nearly all the slate quarries in this region are merely deep holes in the rock with vertical sides, most of the working being done at the bottom of the hole. The Albion Quarry may be taken as the largest in the Pen Argyle region, it being about 300 x 500 feet in horizontal area and nearly 250 feet deep. The photographic view in Plate XLVII shows the upper part of the north side of this quarry, upon which some work is in progress at the depths of about 60 and 120 feet below the surface. The curved ribbons can be plainly seen running diagonally across the view, the one which is very prominent being a quartz vein. The planes of cleavage in the foreground are closely horizontal. Crossing the picture are seen the ropes of the cable derricks by which the slate is hoisted and run out to the banks.

The photographic view of the Old Bangor Quarry, at Bangor, Pa., in Plate XLVIII, is taken looking toward the northeast. It clearly shows the manner of working in benches whose surfaces are parallel with the plane of cleavage. As the benches are worked downward, the top is being constantly stripped off in order to allow new ones to be started, and thus the horizontal extent of the quarry is continually increased, its maximum depth remaining at about 125 feet below the original surface of the ground. This method of working in benches is peculiar to this particular quarry and can only be pursued when the uncovered area is very large. There are five of these benches, each being about 15 feet in height and from 30 to 40 feet in width. The curved ribbons showing the planes of original stratification can be
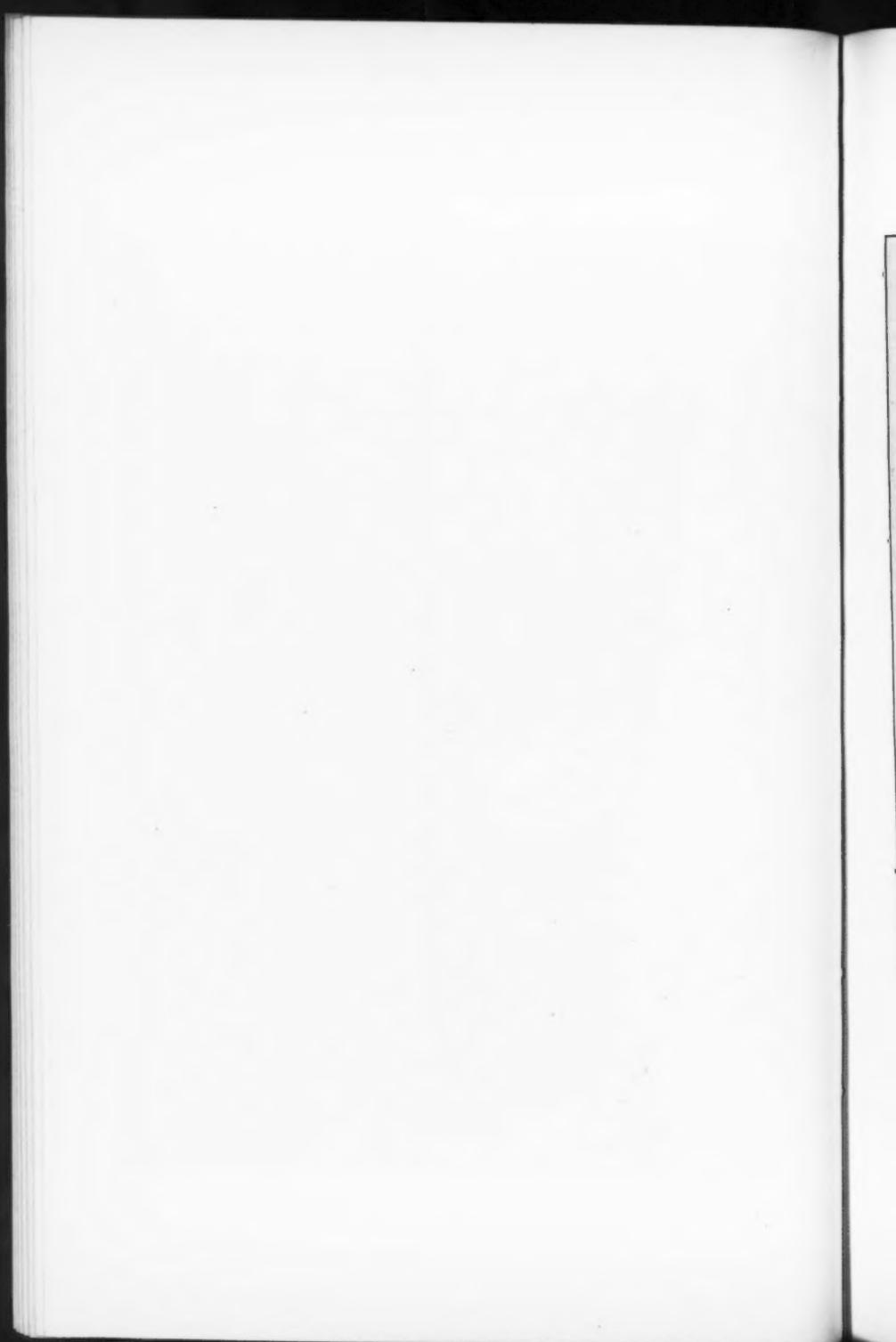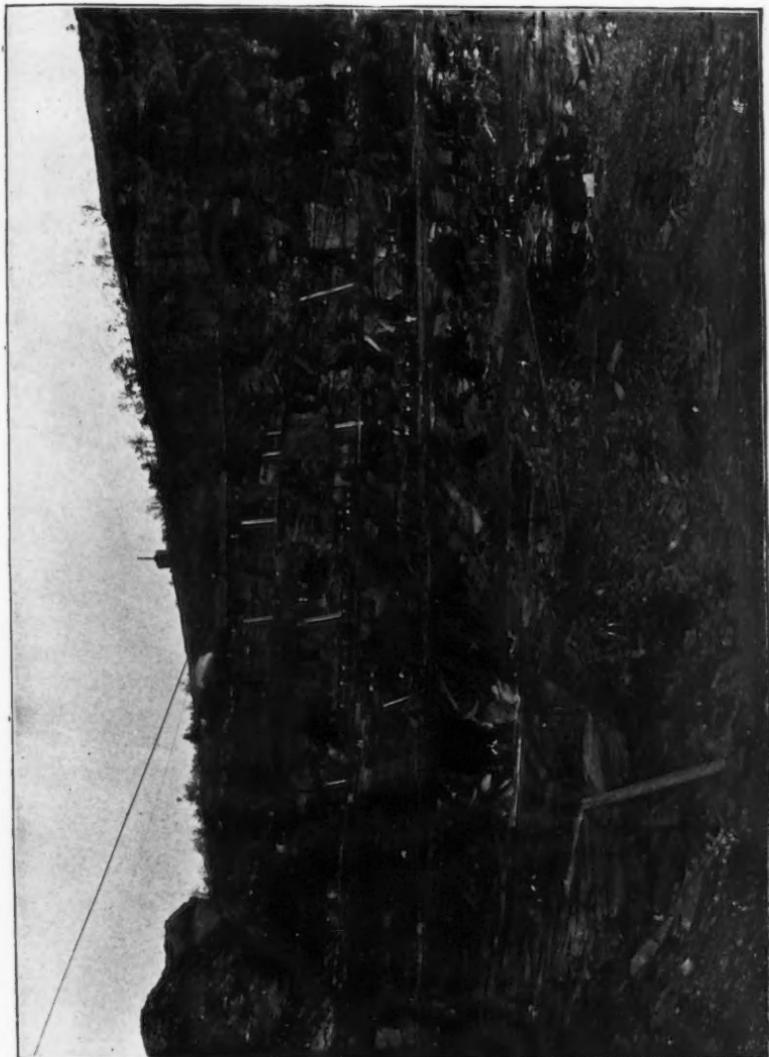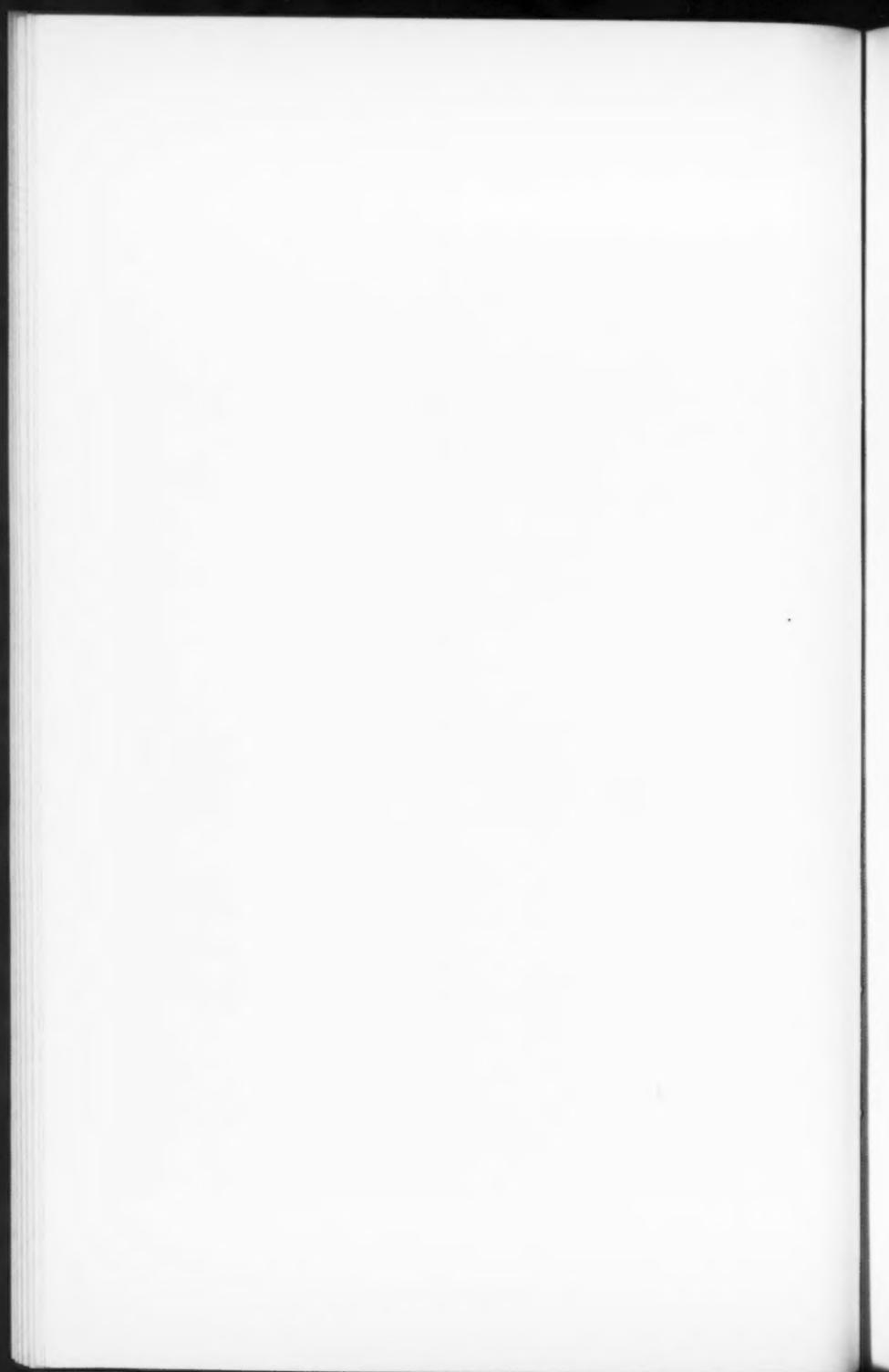
faintly seen, the general direction of the curves being somewhat similar to those in Fig. 2. This is the oldest quarry in the Bangor region, having been opened in 1866, and the horizontal uncovered area is now about 400 x 900 feet.

Roofing Slates.—In the manufacture of roofing slates everything is done by hand except dressing the edges. The blocks delivered at the shanties are first split into thicknesses of about 2 inches. These are piled up in a shanty beside a workman called a splitter, who, with a wooden mallet and a long, thin chisel, divides each into halves, and continues the process until they are reduced to the required thickness of about $\frac{3}{16}$ths of an inch. He then cuts out pieces in approximate sizes, and these are taken by an assistant and squared off into regular shape and size on a dressing machine. The rock of some quarries requires to be kept damp from the time it is first exposed until it is split into the final sizes, otherwise the cleavage becomes difficult.

Plate XLIX is a view of one of the slate banks of the Albion Quarry at Pen Argyle, where a number of shanties and workmen with their tools are seen. On the extreme right is a large block being drawn up on a car, and a pile of finished slates is seen on the left near the engine house. The rude awnings of boards and branches protect the men from the glare and heat of the summer sun, which on a slate bank always seems more oppressive than elsewhere. The workmen of this quarry are mostly English, while at some other quarries Welshmen predominate. It seems a very simple matter to split slate, but in reality it is a trade which requires much experience.

Roofing slates are made in numerous sizes, from 14 x 24 inches down to 6 x 12 inches, the longer dimension being that which is placed parallel with the rafters of the roof. In all roofing which is properly done, a triple lap of 3 inches is allowed ; thus, for a 24-inch slate 10½ inches are exposed, 10½ inches are covered by the slate above it, and 3 inches are covered by two slates above it. The amount of slate required to cover a space 10 x 10 feet in this manner is called a square which is the unit by which they are sold. For slates 12 x 24 inches it takes 114 to make a square; for those 8 x 16 inches, 277 make a square, and so on.

The normal product of roofing slates is called No. 1 stock, and this is entirely free from ribbons. Some quarries make second and third quality slates, called No. 1 Rib and No. 2 Rib, respectively, the

former containing ribbons near one end only, so that when laid on the roof, the exposed parts are free from them. The color of the slate produced in the quarries at Pen Argyle and Bangor, which are those here specially investigated, is a permanent dark bluish grey. The
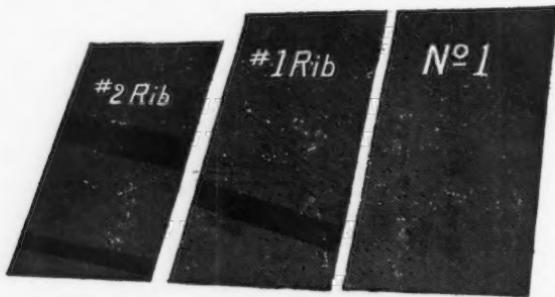

Fig. 3.

color of the ribbons, however, is nearly black at first, yet on exposure to the weather they exhibit a whitish efflorescence and soon show signs of crumbling and decomposition. This is due to the sulphide of iron which they contain, and also to their porosity and softness. Although the price of these inferior qualities is from 20 to 40 per cent. less than that of the standard stock, they should not be allowed on a roof which is to be regarded as a good piece of work.

The roofing slates which form the subject of this paper are those from the Albion Quarry at Pen Argyle, Pa., and the Old Bangor Quarry at Bangor, Pa. The Albion Quarry is near the top of the slate formation, and on the section in Fig. 1 would be projected near the point marked *A*. The Old Bangor Quarry is about 3 miles from the Blue Mountain near *B* in Fig. 1, and thus at the bottom of the upper or softer slate beds. These two quarries are the largest in their respective regions, the entire product of the former, and a large part of that of the latter, being roofing slates which by general reputation stand high in quality.

The investigations of the properties of slate which are found on record are few, and these are almost entirely by chemical analysis. Silica and alumina are supposed to give strength and toughness, the carbonates of lime and magnesia are liable to be acted upon and washed out by the rain, and the compounds of iron and sulphur are

known to promote disintegration under the action of smoke and acid fumes. Something as to quality can therefore be judged by the comparison of chemical analyses, but the information thus obtained is so slight as to have little weight with an engineer, particularly when he considers that the mineralogists inform us that rocks of the same identical chemical composition may have quite different properties on account of different lithiological structure. A chemical test of iron or steel affords but little information to the engineer concerning its physical properties, and he demands that quantitative results concerning its toughness and strength must be known. So it should be with stones and slates.

Method of Investigation.—It was purposed in investigating these roofing slates, to experiment as closely as possible upon those properties which are called into service in resisting the stresses to which they are subject, and upon those qualities which either assist or resist the disintegrating action of the atmosphere and weather. The strength and toughness of slate are important elements in preventing breakage in transportation and handling, as well as in resisting the effect of hail, or of stones maliciously thrown upon the roof. They are also brought effectively into play by the powerful stresses produced by the freezing of water around and under the edges. Porosity, on the other hand, is not a desirable property, for the more water the slate absorbs, the greater the disintegrating action when it freezes and thaws. Density is a quality of value, for, in accordance with a fundamental principle of the science of the resistance of materials, the greater the specific gravity of one of several similar substances, the greater is its strength. Hardness may or may not be a desirable quality according as it is related to density or to brittleness. Lastly, corrodibility, or the capacity of being disintegrated by the chemical action of smoke and of fumes from manufactories, is certainly not desired in roofing slates.

The investigation was undertaken primarily to compare the qualities of the Pen Argyle and Bangor slates in a scientific manner. In order to do this, such tests were selected as seemed likely to be both precise and simple, and of such a character that quantitative results concerning each of the above properties could be determined. These results will be given below for a number of specimens, and they will be discussed and compared with the view of ascertaining the relation between

the different properties. Lastly, by the help of chemical analyses, the relation of the physical qualities to the presence or absence of certain elements is to be studied.

The pieces tested were 12 inches wide by 24 inches long, and varied in thickness from $\frac{3}{16}$ths to $\frac{1}{4}$ of an inch. They were all free from ribbons and were presumably of the best product of the respective quarries. From the Albion Quarry at Pen Argyle, there were twelve specimens marked A1, A2, A3, etc. From the Old Bangor Quarry at Bangor, there were also twelve specimens, marked B1, B2, B3, etc. These letters and numbers were kept upon the specimens and their fragments throughout the different tests, thus enabling the different properties to be compared for each individual specimen. The general appearance of the slates was very similar, except that the B pieces were somewhat more even in surface and slightly darker in color.

Strength.—The transverse or flexural strength of the slates was selected for experiment because of the ease and accuracy with which the tests can be made, and also because such stresses are brought upon it in actual use rather than those of pure tension or compression. The pieces were supported in a horizontal position upon wooden-knife edges 22 inches apart, and then loads were placed upon another knife edge half way between the supports. This load was applied by means of sand running out of an orifice in a box, at the uniform rate of 70 pounds per minute, and by the help of an electric attachment the flow of sand was stopped at the instant of rupture. The slates were always placed upon the supports, so that the beveled edges were on the lower side. As the load was increased, the deflection of the slate could be observed upon a scale and the ultimate deflection was recorded.

The strength of a beam or plate broken in this manner is indicated by the modulus of rupture which is the computed horizontal rupturing stress on the upper and lower fibers, and is intermediate in value between the ultimate tensile and compressive strength. Let W be the load in pounds which causes rupture when applied at the middle, let l be the distance between the supports in inches, b the breadth and d the thickness of the plate in inches. These being carefully found by observation, the formula

$$S = \frac{3\,Wl}{2bd^2}$$

furnishes the means of computing the modulus of rupture S, whose value is then in pounds per square inch.

In Tables A and B are given the values of the modulus of rupture for each of the twenty-four specimens. In four cases this value is the average of two tests, the second one being made upon pieces 10 inches long which were cut from the larger broken specimens. The figures for these duplicate tests will be interesting as showing the slight range in the results, and thus establishing the accuracy and value of this method of investigation. They give·the following values for the modulus of rupture in pounds per square inch—

Mark of specimen	B3	B7	B9	B11
S for large slate	9 750	8 420	10 195	8 100
S for small slate	9 720	8 450	10 235	8 140

and the average of these is stated in Tables A and B, while for the other specimens tests were made upon the larger sizes only.

The mean value of the modulus of rupture of all the specimens is seen to be 7 150 pounds per square inch for the Albion slates, and 9 810 pounds per square inch for the Old Bangor slates.

Toughness.—The ultimate deflections of the pieces broken under transverse stress furnish an indication of their toughness, in the same manner that the ultimate elongation of a metallic specimen under tensile strain is an index of toughness and extensibility. The greater the ultimate deflection of a bar, the less is its brittleness, and the greater its toughness, other things being equal. All the specimens of slate were so elastic that the deflection of the middle part, where the load was applied, could easily be noted on a scale by the eye, with an error rarely exceeding $\frac{1}{32}$d of an inch. The test can, therefore, be readily made with the simplest apparatus. The results for the individual specimens are seen in Tables A and B, all of which were found from the pieces laid upon supports 22 inches apart. The mean values of the ultimate deflection are 0.270 inches for the Albion and 0.313 inches for the Old Bangor specimens.

The toughness of slate can also be inferred from the manner of rupture of the specimens. It was observed in general that the A pieces broke squarely across, while a B piece ruptured in different planes on the upper and lower surfaces; so that, in fact, it often split or sheared into two thinner sheets, which were then pulled apart from each other. The structure of the Old Bangor specimens was, hence, more laminated and fibrous than that of the Albion ones.

Specific Gravity.—The density was next determined for each specimen by weighing it in air, and then in water, from which data the specific gravity was computed.

The results given in Tables A and B show that the mean density of the two kinds of slate is almost exactly the same, being 2.775 for the Albion and 2.780 for the Old Bangor, but the former seem the more uniform in density as shown by the smaller range between the maximum and minimum values.

Softness or Capacity for Abrasion.—The hardness of the slates was next tested by subjecting them to abrasion upon a grindstone whose thickness was $1\frac{1}{2}$ inches and diameter 7 inches. A piece of slate about 4 x 4 inches was accurately weighed, and then held against the grindstone by a lever, which exerted a constant pressure of 10 pounds upon it while the grindstone was turned fifty times. The piece was then weighed again, and the difference of the two weights gives the amount ground off. The results thus obtained indicate the hardness, or rather the softness, for the greater the abrasion, the softer is the material. Tables A and B, which give these results for each specimen, show the mean values to be 80 grains for the Albion and 128 grains for the Old Bangor slates. It would appear, therefore, that if used for sidewalks, or in circumstances where hardness is the only important quality, the Albion slate is the better of the two kinds. For a roofing material, however, hardness is not necessarily a valuable quality, and it will appear later in this discussion that the softer slate has the higher strength and weathering qualities.

Porosity.—The well-known test for porosity is to determine the percentage of water absorbed by the specimens under similar conditions. In the absence of standard rules regarding the shape and size of the specimen, its degree of dryness, or time of immersion, the following procedure was adopted: A piece of slate from each specimen was cut to a size about 3 x 4 inches, the edges being left rough and irregular. These were dried for twenty-four hours in an oven at a temperature of 135 degrees Fahr. After cooling to the normal temperature of the room, they were weighed on delicate scales, and then immersed in water for twenty-four hours, when they were taken out and weighed again. The difference of these weights, divided by the first weight, gives the percentage of water absorbed by a specimen. Tables A and B exhibit the results thus obtained, and it is seen that

the mean percentage for the Albion slate is 0.238, while for the Old Bangor slates it is 0.145.

Corrosion by Acids.—In order to imitate the action of smoke and of sulphurous fumes of manufactories, the following test was used : A solution of hydrochloric and sulphuric acids was prepared, consisting by weight of 98 parts of water, 1 part of hydrochloric and 1 part of sulphuric acid. In this, pieces of slate about 3 x 4 inches in size were immersed for certain periods of time, having first been carefully weighed. After being taken out of the solution, they were dried for two hours in the normal air of the laboratory, and were again weighed. Thus the loss of weight due to the corrosive action of the acids was ascertained. The results were then transformed into percentages of the original weight, which give an absolute measure of the corrosion. Two specimens of each kind of slate were kept for twenty-four hours in the solution, and gave the following percentages of loss of weight :

Albion.		Old Bangor.	
A6	.342	B5	0.297
A8	0.400	B7	0.235
Mean	0.371	Mean	0.266

Two specimens of each variety of slate were kept for eighty-seven hours in the acid solution, and there were found the following percentages of loss of weight :

Albion.		Old Bangor.	
A5	0.676	B2	0.633
A7	0.627	B12	0.696
Mean	0.651	Mean	0.665

Most of the specimens, however, remained in the acid for a period of sixty-three hours, and from these the percentages of loss of weight were obtained, which are given in Tables A and B. After the tests, the surfaces of the pieces showed but slight traces of acid action, notwithstanding the loss of weight.

These results plainly indicate that the corrosion or disintegration rapidly increased with the time. The sum of these means for twenty-four hours, sixty-three hours and eighty-seven hours, is 1.569 per cent. for the Albion specimens, and 1.377 per cent. for the Old Bangor specimens, which may, perhaps, be taken as the final values for comparison, although it should be noted that in the long test of eighty-seven hours the Albion mean is less than the Old Bangor mean.

TABLE A.—Tests of Albion Slates.

Mark of the Specimens.	Strength. Modulus of rupture, in pounds per square inch.	Toughness. Ultimate deflection, in inches, on supports 22 inches apart.	Density. Specific gravity.	Softness. Amount in grains, abraded by 50 turns of a small grindstone.	Porosity. Per cent. of water absorbed in 24 hours.	Corrodibility. Per cent. of weight lost in 63 hours in acid solution.
A1	6 920	0.25	2.770	86	0.238	0.588
A2	7 560	0.31	2.775	97	0.219	0.619
A3	6 230	0.25	2.782	65	0.171	0.500
A4	7 330	0.34	2 761	76	0.259	0.614
A5	6 920	0.31	2.781	86	0.303	......
A6	7 130	0.25	2.775	86	0.274	......
A7	9 110	0.31	2.781	86	0.246	......
A8	6 220	0.22	2.775	76	0.197	......
A9	7 565	0.26	2.764	65	0.263	0.616
A10	7 845	0.30	2.765	86	0.228	0.508
A11	6 930	0.22	2.781	86	0.232	0.491
A12	6 080	0.22	2.785	65	0.228	0.438
Means....	7 150	0.270	2.775	80	0.238	0.547

TABLE B.—Tests of Old Bangor Slates.

Mark of the Specimens.	Strength. Modulus of rupture, in pounds per square inch.	Toughness. Ultimate deflection, in inches, on supports 22 inches apart.	Density. Specific gravity.	Softness. Amount in grains, abraded by 50 turns of a small grindstone.	Porosity. Per cent. of water absorbed in 24 hours.	Corrodibility. Per cent. of weight lost in 63 hours in acid solution.
B1	11 550	0.32	2.816	151	0.131	0.410
B2	11 540	0.38	2.807	140	0.170	......
B3	9 740	0.31	2.795	76	0.099	0.439
B4	8 650	0.31	2.784	119	0.127	0.481
B5	7 280	0.25	2.779	86	0.204	......
B6	9 220	0.31	2.759	130	0.174	0.374
B7	8 440	0.27	2.776	119	0.167	......
B8	11 570	0.40	2.769	130	0.123	0.551
B9	10 215	0.32	2.782	173	0.123	0.464
B10	10 900	0.37	2.767	86	0.169	......
B11	8 120	0.23	2.769	140	0.099	0.404
B12	10 490	0.32	2.754	184	0.152	......
Means.....	9 810	0.312	2.780	128	0.145	0.446

Discussion of the Tests.—The above physical tests were mostly made under the writer's direction, by Mr. J. P. Brooks, Instructor in Civil Engineering in Lehigh University. Great care was taken to conduct them so that the numerical results would be strictly comparable.

The recapitulation of the mean results of these tests will be useful at the outset of the discussion.   These mean values have, according to the principles of the method of least squares, about twelve times the weight of a single determination, and should hence be expected to furnish a tolerably reliable indication concerning the properties under investigation.   They are given in Table C, and it is at once seen that

TABLE C.—MEAN RESULTS OF PHYSICAL TESTS.

Property.	Measured by—	Albion slates.	Old Bangor slates.	Mean of both.
Strength	Modulus of rupture, in pounds per square inch..........................	7 150	9 810	8 480
Toughness.	Ultimate deflection, in inches, on supports 22 inches apart.................	0.270	0.313	0.291
Density........	Specific gravity........................	2.775	2.780	2.777
Softness..	Weight, in grains, abraded on grindstone under the stated conditions...	80	128	104
Porosity........	Per cent. of water absorbed in 24 hours, when thoroughly dried..............	0.238	0.145	0.191
Corrodibility...	Per cent. of weight lost in acid solution in 63 hours..........................	0.547	0.446	0.496

the different qualities are connected by definite relations, the strongest slate being the toughest and softest, as also the least porous and corrodible.

This conclusion is established in a manner thoroughly satisfactory by plotting the results for the individual specimens. For instance, in Fig. 4 is given a comparison of the A and B specimens as respects strength and porosity, the values of the former being laid off as ordinates, and those of the latter as abscissas. It is seen that the general results concluded from the means hold good, likewise, for many of the specimens; for example, B5 is the weakest Old Bangor specimen, and its porosity is the greatest.

Fig. 5 exhibits the relation between strength and softness in a similar manner, the conclusion being everywhere apparent that the greater the strength, the greater, also, is the softness or capacity for abrasion.   This result was unexpected, and cannot, of course, be laid down as a general rule applicable to building-stones, or even to slates which differ greatly in structure.   The capacity for abrasion here seems allied to toughness, or it denotes the lack of brittleness; but this relation certainly is not a general one, although it is true for the roofing slates here discussed, and also for others which the writer has

investigated. The testimony of the quarrymen, moreover, as far as he has been able to obtain it, seems to verify the conclusion that in this slate region, softness, strength and toughness are qualities closely connected.
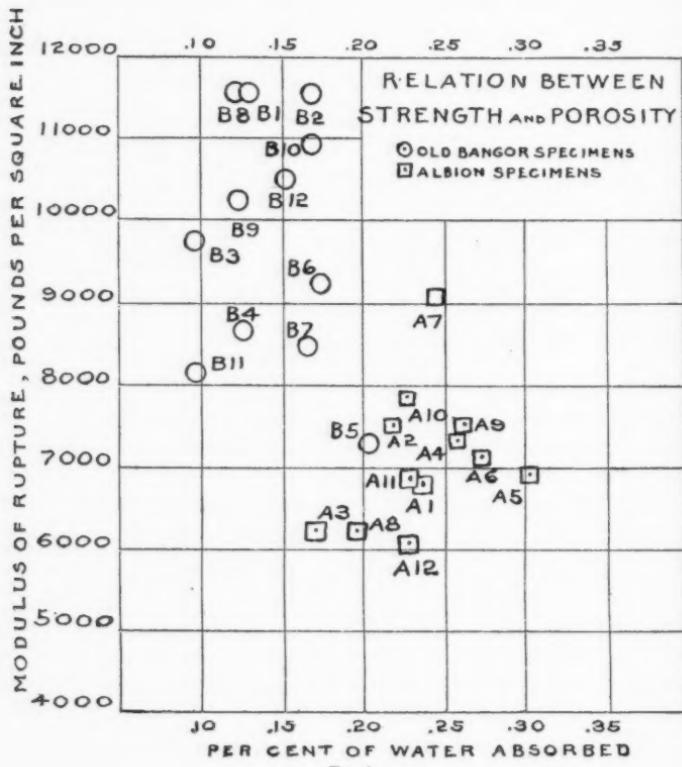

Fig. 4

Similar graphic comparisons have been made between strength and toughness, strength and corrodibility, and porosity and corrodibility. The conclusion is irresistible that these qualities are so connected that one may be taken as an approximate index of the others; and, after carefully considering the whole field, the writer does not hesitate to decide that the test for transverse strength is the one which is the most satisfactory for roofing slates, if only one test is to be made. It is one

that can be made quickly, and without expensive apparatus. The modulus of rupture is an absolute quantity independent of the size of the specimen, and it gives to the engineer a more definite idea of the quality of materials than do the figures indicating other properties. In making this test it will usually be easy to measure the deflections and thus obtain the means for comparing the toughness as well as the strength.
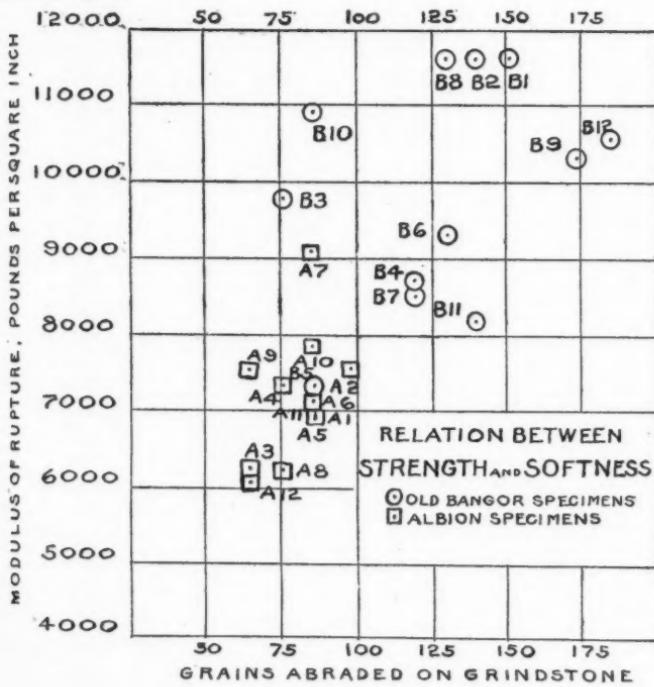

Fig. 5.

While the test for porosity is a valuable one, it is necessary that the specimens should be brought to the same degree of dryness by heating them for an entire day; and as the amount of water absorbed is only about one-fifth of 1 per cent., the weighing must be done with care. The test for corrodibility is also a good one, but precise weighing is also necessary. The tests for abrasion on a grindstone and for specific gravity appear lowest of all in practical value, and their use

cannot, in general, be recommended, the former being liable to be mis-
interpreted, and the latter not being sharply related, in this case at
least, to the strength and weathering qualities.

Chemical Analyses.—Although the interpretation of chemical analyses
is, at the best, imperfect and unsatisfactory, it was not thought advis-
able to entirely neglect them in this investigation.   The physical
properties being known, they may perhaps be in some measure ac-
counted for by the chemical composition.   Hence, an analysis was
made of each of the varieties of slate by Dr. Frederick Fox, Instructor
in Chemistry in Lehigh University.   The expense of chemical work is
so great that an analysis of each specimen could not be undertaken,
and accordingly B3 was selected as being an average sample in strength
and weathering qualities.   The analysis of the A slates was begun
before the physical tests were made, and accordingly small pieces
broken from the corner of the A1 and A2 specimens were ground up
and mixed together as a mean compound.   The complete analysis of
the slates was not undertaken, this being a long and expensive opera-
tion, but such elements were determined as would probably afford in-
dications, first, of its valuable qualities, and, secondly, of its injurious
constituents.   The following is the report of the chemical analyses.

	Albion.	Old Bangor.
Silica (SiO)..................................	.55.18	56.97
Oxides of iron and aluminum ($Fe_2O_3 + Al_2O_3$)...	25.57	26.05
Carbonic acid (CO_2) and organic matter........	8.36	7.14
Oxide of calcium (CaO) or lime................	4.09	4.38
Oxide of magnesium (MgO)	2.10	2.69
Sulphur (S)..................................	0.700	0.462
Oxides of sodium and potassium (by difference).	4.00	2.31
Manganese is present in all the slates.		

The above are the substances actually determined in the chemical
work.   They do not, however, exist in these forms in the slate itself.
Slate consists principally of silicates of iron and aluminum, with
smaller proportions of silicates of sodium and potassium, together
with carbonates of lime and magnesium, and sulphide of iron as im-
purities.   In the analysis the silicates are broken up into silica and
oxides, and these separately determined.   So the carbonate of lime is
broken up into carbonic acid and oxide of calcium, and the latter de-
termined.

Now, the valuable constituents of slate are the silicates of iron and aluminum, or the clay, this being inert and incorrodible. The percentage of this is easily computed from the chemical analyses to be as follows:

	Albion.	Old Bangor.
Silicates of iron and aluminum	80.75	83.02

which indicates the Old Bangor to be better than the Albion, other things being equal.

The injurious constituents in slate are the carbonates of lime and magnesia, and the sulphide of iron, or iron pyrites. The carbonates are easily attacked and dissolved by water which has been rendered slightly acid by smoke, and the pyrites is apt to cause disintegration. The chemical analyses furnish the means of computing the carbonates and the amount of pyrites is probably closely proportioned to that of the sulphur. The percentage of sulphur, however, is so small in all the specimens, that its influence is probably slight. The computed percentages of carbonates are—

	Albion.	Old Bangor.
Carbonate of lime	7.40	7.82
Carbonate of magnesia	4.41	5.65
Total carbonates	11.71	13.47

and these indicate the Albion as likely to be less corrodible than the Old Bangor slate. This conclusion is not justified by the corrosion tests, except in the case of the two specimens which were soaked for eighty-seven hours.

In the presence of the physical tests, the chemical indications as to liability to corrodibility may, perhaps, be explained by saying that in its constituents the Albion has the greater capacity to resist corrosion, but that its structure, as shown by strength and porosity, is such as to render it scarcely equal to the Old Bangor in this respect.

Conclusions.—The above investigation seems to indicate the following conclusions regarding the soft roofing slates of Northampton County, Pa.:

1. Slates containing soft ribbons are, by common consent, of an inferior quality, and should not be used in good work.

2. The soft roofing slates weigh about 173 pounds per cubic foot,

and the best qualities have a modulus of rupture of from 7 000 to 10 000 pounds per square inch.

3. The stronger the slate, the greater is its toughness and softness, and the less is its porosity and corrodibility.

4. Softness, or liability to abrasion, does not indicate inferior roofing slate; but, on the contrary, it is an indication of strength and good weathering qualities.

5. The strongest slate stands highest in weathering qualities, so that a flexural test affords an excellent index of all its properties, particularly if the ultimate deflection and the manner of rupture be noted.

6. The strongest and best slate has the highest percentage of silicates of iron and aluminum, but is not necessarily the lowest in carbonates of lime and magnesia.

7. Chemical analyses give only imperfect conclusions regarding the weathering qualities of slate, and they do not satisfactorily explain the physical properties.

8. Architects and engineers who write specifications for roofing slate will probably obtain a more satisfactory quality if they insert requirements for a flexural test to be made on several specimens picked at random out of each lot.

9. Although the field of this investigation is probably not sufficiently extended to fully warrant the recommendation, it is suggested that such specifications should require roofing slates to have a modulus of rupture, as determined by the flexural test, greater than 7 000 pounds per square inch.

DISCUSSION.

CHARLES B. BRUSH, Vice-President Am. Soc. C. E.—I want to congratulate the gentleman on the very pleasing way in which he has presented his paper. Instead of reading the paper in full, or instead of making certain selections from it which are not connected with other selections, he has taken the gist of the paper and presented it to us.

There is one thing in relation to slates that I have not heard Mr. Merriman refer to. A very common test is simply to balance one of the slates on your hand and strike it and listen to the ringing sound. So far as my experience has gone, this is a very simple and it has proved to be a very satisfactory test. I do not know what his expe-

rience has been in relation to that test, whether it is a fallacy or an indication of the good quality of the slate.

Another very important quality is the degree of brittleness. I presume that was covered by what the paper presents in relation to toughness and softness. Of course, a slate readily broken when punched is of much less value than one which can be readily fastened in place. I was not present at the beginning of the meeting, however, and did not hear the early part of the paper. Perhaps Mr. Merriman may have referred to these two very common tests; if not, I would be glad if he would inform us as to the results obtained by common practice as compared with the more elaborate tests he has presented.

MANSFIELD MERRIMAN, M. Am. Soc. C. E.—In regard to the ringing test, I know that it is one often used with slates as with bricks, but as its results cannot be expressed quantitatively I did not make notes upon it. The question of brittleness is mentioned in the paper, this quality being the reverse of toughness. Slates which break with a small deflection and show a square fracture are apt to be brittle.

Mr. BRUSH.—The A slate, as I take it, is the inferior; the B is that taken from the center of the bed, or the better; and the C slate the best, if I understand the gentleman. If that is true, I would like to ask whether any similar examinations have been made in connection with the bottom slate—the C slate ?

Mr. MERRIMAN.—There are few quarries in the C beds, and I know of only one which makes roofing slate. This slate is very heavy, and some of it is said to weigh more than 200 pounds per cubic foot. It is not so easily split into thin plates as the A and B kinds, and hence comparatively little roofing slate is produced from it. I have made no tests of the C slates and do not know whether they are better than the A and B kinds, or not.

Mr. BRUSH.—What is that C slate used for principally ?

Mr. MERRIMAN.—Largely for sidewalks and for steps or risers. It is usually too hard for school slates or blackboards.